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COST AND FEASIBILITY EVALUATION FOR THE EXCAVATION OF LARGE HEMISPHERICAL CAVITIES IN RAINIER MESA

Engineering Decision Analysis Company, Inc.
2400 Michelson Drive
Irvine, California 92715

October 1978

Topical Report for Period January 1978—October 1978

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20. ABSTRACT (Continue on reverse side if necessary and identify by block number)

In order to provide necessary facilities for the fielding of experiments in support of the underground nuclear testing program, there has been an increase in interest concerning the construction of large underground caverns in Rainier Mesa. As a result, a cost and feasibility program was commissioned to evaluate hemispherical cavities between 24.4 and 91.4 m (80 and 300 ft) in diameter. The rock support designs for the cavities are based upon the use of internally installed rockbolts for the smaller cavities and tendons installed from annular galleries for the larger chamber. The evaluation program (over)

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20. ABSTRACT (Continued)

included research into past experience pertaining to the excavation and support of large underground caverns, assessment of the geological conditions which exist in Rainier Mesa, preliminary design of the cavities and their rock support system based upon the geological setting, development of mining plans for the excavation of the caverns, and estimation of the cost and manpower schedule to complete the construction of each size chamber based upon the preliminary design and mining approach. This report presents the findings of the cost and feasibility evaluation program and includes recommendations concerning the maximum practical chamber size considering schedule, dollar, and state-of-the-art constraints.

PREFACE

In order to provide necessary facilities for the fielding of experiments in support of the underground nuclear testing program, the Defense Nuclear Agency (DNA) commissioned a cost and feasibility evaluation program pertaining to the design and construction of large underground cavities. This report which documents the results of this feasibility evaluation was prepared by Engineering Decision Analysis Company, Inc. (EDAC) as part of its contract (DNA001-78-C-0281) to provide structural evaluation and design support for the Underground Nuclear Testing Program.

This large cavity program involved the bringing together of experts in the fields of large cavity design, large cavity construction at the Nevada Test Site (NTS), and geology of the NTS in order to provide input toward this effort. EDAC acted as program coordinator for this group of experts and wishes to acknowledge the valuable assistance of those who have contributed textual material, data, and information toward the preparation of this report. Specifically EDAC would like to thank the following individuals who have been extremely helpful and responsive to all our inquiries.

Individual	Affiliation
Scott Butters	Terra Tek
Rod Carroll	United States Geological Survey
Edward Cording	University of Illinois
Bill Ellis	United States Geological Survey
Bill Flangas	Reynolds Electrical & Engineering Co.
Dan Koss	Fenix & Scisson
Joe LaComb	Defense Nuclear Agency
Andrew Merritt	Deere-Merritt, Inc.
Robert Pritchett	Reynolds Electrical & Engineering Co.
Larry Skousen	Department of Energy
Susan Steele	United States Geological Survey
Don Waltman	Fenix & Scisson
Gerry Woodard	Holmes & Narver

Section 2 contains the geological setting, lithology and material property values characteristic of the Rainier Mesa tuff. Technical material for this section was provided by Susan Steele, Bill Ellis and Rod Carroll of the USGS and by Scott Butters of Terra Tek.

Section 3 and 4 document the past experience related to large cavity excavation and describe the general design considerations respectively. Material for these sections was essentially written by Edward Cording of the University of Illinois and Andrew Merritt of Deere-Merritt, Inc.

Sections 5 and 7 present the specific design detail for each of the cavities and the associated costs of construction and required schedule, respectively. The information presented is based upon design layouts and cost and schedule estimates developed by Dan Koss and Don Waltman of F&S with input on mining and excavation methods from Bill Flangas and Bob Pritchett of REECO and Larry Skousen of DoE.

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1. INTRODUCTION

1.1 PURPOSE AND SCOPE OF EFFORT

Measurement of certain data associated with nuclear testing necessitates the reception of waves at the instrument location which are unaffected by reflection off internal structures or tunnel surfaces. Therefore, in order to provide necessary facilities for the fielding of experiments in support of the underground nuclear testing program, the Defense Nuclear Agency (DNA) commissioned a cost and feasibility evaluation pertaining to the design and construction of large underground cavities in Rainier Mesa. This report was prepared to document the results and conclusions of this feasibility evaluation.

The effort includes research into past experience pertaining to the excavation and support of large underground caverns, assessment of the geological conditions which exist in Rainier Mesa, preliminary design of the chambers and their rock support systems based upon the geological setting, development of mining plans for the excavation of the caverns, and estimation of the cost and manpower schedule to complete the construction of each size chamber based upon the preliminary design and mining approach. Each of these details are addressed in separate sections of this report.

The feasibility investigation was intended to be preliminary in nature and is based upon the use of preliminary analytical evaluation techniques. Similarly the inferred geological setting of a hypothetical site is based upon data obtained from drill holes located in the near vicinity. It is obvious that this level of geological investigation and engineering design analysis are not sufficient for the final design of the chamber and its rock support systems and therefore the additional geological and engineering investigations needed for final design are also outlined in this report.

1.2 PROBLEM DESCRIPTION

The following are various aspects of the problem statement given to the evaluation team which form the basis for the considerations addressed in this report.

Site - The large cavity evaluation program is intended to be generic in nature so as to be applicable to a wide range of sites in Rainier Mesa. However, for purposes of defining a general geological setting for the study, the general geology of a site located between the U12e.07 and U12e.18 drifts in the E-tunnel complex is inferred. The hypothetical location is shown in relation to the various Rainier Mesa tunnel systems in Figure 1-1.

Lithological Location - For purposes of the evaluation, the cavity location is assumed to be within tunnel beds #4 with the cavity invert level approximately centered at the interface between tunnel beds 4G and 4H. Based upon preliminary considerations, these lithological layers appear to be the "best" level within Rainier Mesa for a large underground cavity. Therefore the conclusions in this report are only directly applicable for cavities situated within tunnel beds #4 and may be somewhat over-optimistic for caverns located at other possible levels.

Shape - The chamber is to be assumed to have a horizontal planar floor and its size is to be specified in terms of the minimum radius to the sidewalls or crown of the cavity from the center of the floor invert. A hemispherical opening is the optimum shape from the standpoint of maximizing the minimum radius with the least excavated volume. Variations from the hemisphere are to be evaluated as rock stability considerations dictate.

Support Systems - Two cavity support concepts will be considered. The first involves the use of internally installed rockbolts as the means of primary rock support. The second involves the use of annular tendon galleries located above the chambers from which primary support members can be introduced prior to general excavation.

Size - Cavities ranging in size from 24.4 to 91.4 m (80 to 300 ft) in diameter will be evaluated. Internal support will be considered for cavities with spans of 24.4, 36.6, 48.8, and 54.9 m (80, 120, 160, and 180 ft) while external support will be considered for cavities with spans of 54.9, 73.2, and 91.4 m (180, 240, and 300 ft).

From these basic guidelines, the feasibility of construction, the design of the chamber and its rock support system, and the associated costs and schedule estimates were developed and are presented in this report.

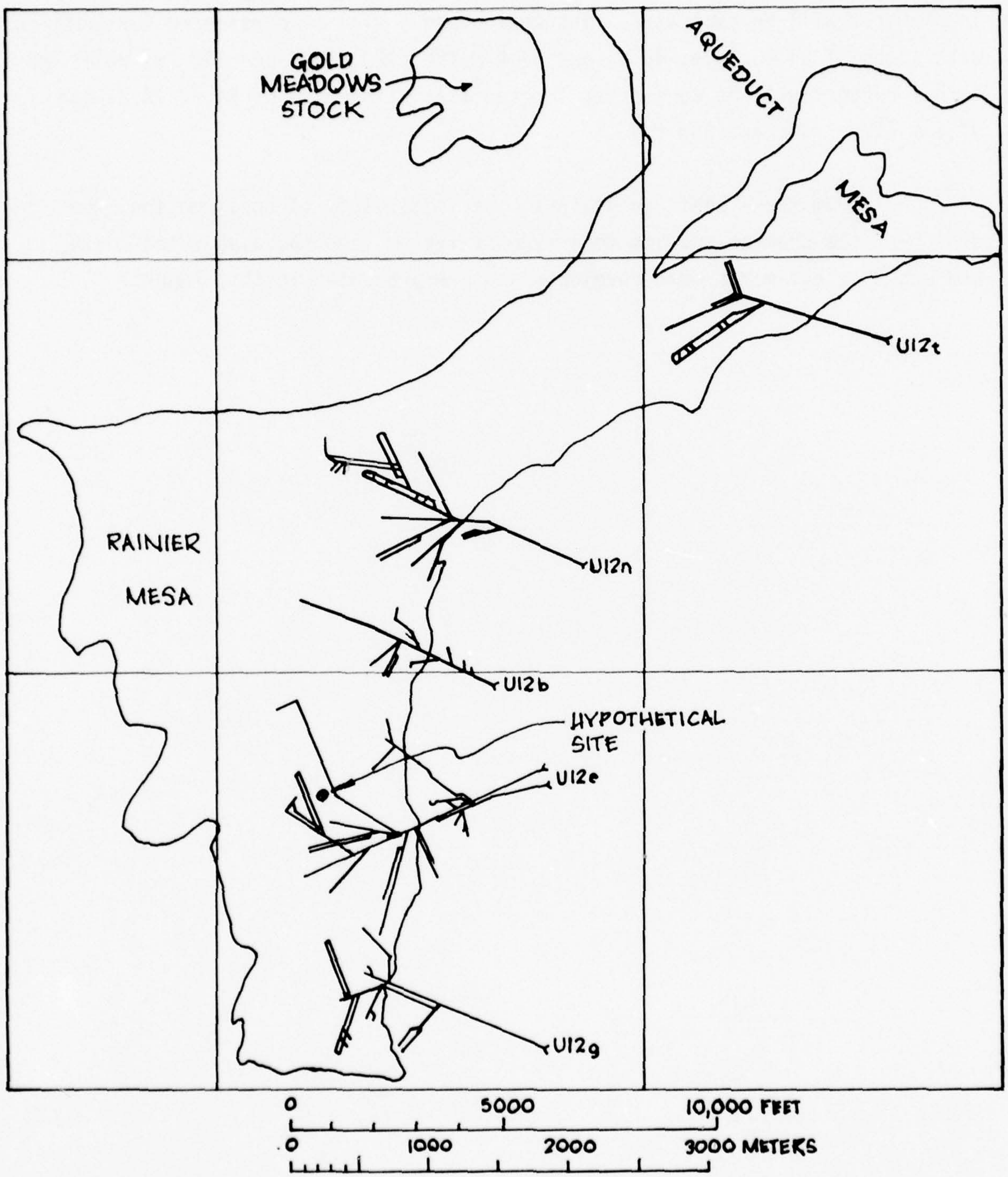


FIGURE 1-1. LOCATION OF HYPOTHETICAL SITE IN RAINIER MESA

2. GEOLOGICAL SETTING

In order to provide a technical basis for the design of the large cavities, a hypothetical site located between the U12e.07 and U12e.18 drifts in the E tunnel complex was selected. A tunnel level detail showing the relationship between the two drift systems and the proposed site is shown in Figure 2-1. The site is located in a region which is reasonably well defined in terms of faults and bedding planes and therefore provides a reliable basis for design assumptions pertaining to the character of the rock mass.

This section of the report describes the overall regional geology of Rainier Mesa and the specific inferred geology of the hypothetical site. In addition, important material property values and the in-situ stress state which is expected to characterize the proposed site are also developed. It should be noted that the geological data presented are inferred from existing drill hole data in the near vicinity and no new geological investigations were conducted as part of this evaluation. It is expected that geologic exploration of any site selected for the excavation of a large cavern will take place as part of the final design effort and the level and types of field investigations which should be performed at the actual site are described in Section 6 of this report.

2.1 REGIONAL GEOLOGY

Generally, the geology of Rainier Mesa consists of layers of Tertiary ash flow, calc-alkaline ash fall, peralkaline ash fall, reworked ash fall, and tuffaceous sandstone units which overlay Mesozoic granite at the northern edge of the Mesa and overlay pre-Tertiary carbonate and clastic rocks at the central and southern portions of the Mesa. A thick, competent, welded ash flow covers the entire Rainier Mesa. The general stratigraphy of Rainier Mesa is shown in Figure 2-2.

In support of other underground nuclear testing programs, two horizontal drill holes (U12e.15 UG-2 and U12e.14 UG-10) and one vertical drill

hole (UE12e #1) were cored and are used to stratigraphically and structurally evaluate the proposed site in the E tunnel complex. These three drill holes are shown in relation to the hypothetical site in Figures 2-3 and 2-4 which depict the surface contours and tunnel level geology, respectively. Additional geological information from these drill holes is presented in Figure 2-5 which shows the pre-Tertiary surface structure and Figures 2-6 and 2-7 which depict the relationship between the various geological layers. The lithologic descriptions of these drill holes, including the thickness of the units penetrated, are included as Appendix A. The geology of the drill holes are graphically shown in Plates 1, 2, and 3.

Referring to Figures 2-10 and 2-11 it can be seen that the proposed site begins in tunnel bed 4G and penetrates up the stratigraphic section to tunnel bed 4K, all of which are of Tertiary age. The pre-Tertiary surface is located approximately 243-274 m (800-900 ft) below the proposed site area (Figure 2-6). The pre-Tertiary rocks under the E-tunnel area consist generally of quartzite on the west and carbonate dolomite on the east with the interface forming a northeast-trending thrust fault (Figure 2-6). The fault is at a very low angle and represents an upper plate moving over a lower one. The probable trace of this thrust fault on the pre-Tertiary surface is located to the west of the proposed site (Figure 2-5) with quartzite constituting the overriding plate of the thrust. The trough of a shallow syncline, which is a concave upward fold in the strata, is also located to the northwest of the site at the tunnel level and can be seen in Figure 2-4 to be in the region where tunnel bed 4K intersects the tunnel level. The water table in the proposed site area is at an approximate elevation of 1524 m (5000 ft).

As mentioned before, the proposed cavity site is located in tunnel beds 4G, 4H, 4J, and 4K with the tunnel level or cavity invert level corresponding to stratigraphic units 4G and 4H. Subunits 4K and 4J consist predominately of massive, zeolitized, calc-alkaline ash fall tuff that contains thin- to thick-bedded, zeolitized peralkaline ash fall tuff. For purposes of evaluating bedded tuffs in Rainier Mesa, a "massive" unit is one which is greater than .9 m (3 ft) in thickness and appears to be homogeneous and free

from internal structure. Subunits 4K and 4J contain varying amounts of swelling and non-swelling clay minerals and, as a result, are soft and contain fewer joints. Argillization is particularly noted in the area associated with the trough of the syncline which should be avoided in the final site selection. Subunit 4H contains thin- to thick-bedded zeolitized calc-alkaline ash fall, reworked calc-alkaline ash fall, and minor amounts of peralkaline ash fall and tuffaceous sandstone. In this area of E tunnel some thin silicified beds appear in the upper part of subunit 4H which tend to be more brittle and to fracture readily. Tunnel bed 4G consists of massive, zeolitized calc-alkaline ash fall tuff that contains numerous thin to thick argillized zones. It should be noted that tunnel bed 3D, which is a calc-alkaline ash fall layer situated stratigraphically immediately below tunnel bed 4, is also considered to be massive and well suited for cavity mining. In addition, the lower subunits of tunnel bed 4 seem to hold up better when exposed to water during mining while the upper subunits tend to be softer and less resistant to water.

Since the Red Hot and Deep Well cavities were mined in G tunnel complex, it is of interest to compare its geology with that of the proposed site. The stratigraphic units encountered in mining G tunnel are the same as those encountered in the E tunnel complex. The tunnel level for the Red Hot/Deep Well area was located in tunnel bed 4G and the cavities extended into the upper tunnel bed 4 region, probably subunit 4J or 4K. Similarly the proposed cavity is expected to extend between the 4G and 4K subunits. The physical and engineering properties of the subunits at both locations are basically the same. The geology of the E and G tunnel systems are shown in Plates 4 and 5 respectively, and indicate the location of mapped fault lines and bedding plane interfaces at the tunnel level. Figures 2-8 and 2-9 depict the general cross-section of the G tunnel area and the geology in the immediate vicinity of the Red Hot and Deep Well cavities. From these figures, it can be seen that in the Red Hot/Deep Well area the major fault lines are generally northwest trending normal faults with dip angles between 70 and 85° and fault displacements on the order of 1.5 m (5 ft). Fault zone spacing in G tunnel is approximately 76 to 91 m (250 to 300 ft). Referring to Plate 4 and Figure 2-4, the major fault zones near the proposed site in E tunnel are also seen to be northwest trending with

angles of dip on the order of 75 degrees. The distance between major fault zones in E tunnel is slightly greater than in G tunnel being approximately 90 to 150 m (300 to 500 ft).

2.2 SPECIFIC INFERRED GEOLOGICAL SETTING

This section establishes the expected geological setting for the proposed site between the U12e.07 and U12e.18 drifts. It is re-emphasized that the conclusions drawn here are based on previously gathered data from core samples and that no new geological investigations were conducted in support of this evaluation.

As mentioned above, the faults in the proposed area trend northwesterly and are spaced between 90 and 150 m (300 and 500 ft) apart (Figure 2-4). Faults are predominately tight and dry with associated fracture zones, if any, averaging less than 1.5 m (5 ft). Silicified rock located near the faults tends to fracture more readily due to brittleness. No faults in the immediate vicinity exceed 7.6 m (25 ft) of displacement and most are less than 1.5 m (5 ft). Most faults mapped at tunnel level are through-going and connect at least two drifts. As seen in Figures 2-6 and 2-7 no faults are thought to extend to the mesa surface with the nearest documented surface structure feature being a photolineation that trends north-northwest over 610 m (2000 ft) from the proposed site area.

Referring to Figure 2-4, it can be seen that the proposed site is located on a northwesterly trending fault block that is bounded on the northeast by a reverse fault with a displacement of 6.1 m (20 ft) and on the southwest by two other northwesterly-trending normal faults with displacements of approximately 1 m (3.3 ft). Bedding planes in the area strike approximately North 15° East and dip an average of 9 to 10 degrees. Assumed cross-sections through the typical tunnel bed #4 site are shown in Figures 2-10 and 2-11 and represent cross-sections C-C' and D-D' identified in Figure 2-4. Since based on current data, the proposed site is relatively fault free, a hypothetical fault with a typical dip and orientation has been added to further typify the fault spacing of an actual potential site as it might be identified after a full geological investigation.

Two faults cut the pre-Tertiary surface below the proposed site as shown in Figure 2-5. The first fault trends northwest with a displacement on the order of 6 to 7.6 m (20 to 25 ft) and is thought to intersect the tunnel level approximately 305 m (1200 ft) south of the site. The second is the CP thrust fault discussed previously which trends north-northeast approximately 61 m (200 ft) northwest of the site and thrusts the older quartzite from the west over the younger dolomite to the east.

The proposed site area is situated in the rock layers of tunnel beds 4F through 4K. Data from the drill holes in the area generally indicate that these rocks are competent and zeolitized. In some areas, tunnel beds 4J and 4K have presented some construction problems because they tend to be weak, contain clay, and slab easily. However, experience gained during the mining of the U12e.18 drift, which passes just to the west of the proposed site, demonstrated that these rock conditions can be controlled by standard support techniques currently employed at the NTS.

During the mining of the U12e.18 drift, water was encountered in the vicinity of the syncline located northwest of the E tunnel site. The concentration of ground water in this depositional low could have caused alteration and weakening of the tuff since, during mining, it was found that the tuff from tunnel beds 4J and 4K was soft and badly air-slacked. However, no severe support problems occurred. It is felt that the ground water encountered was associated with the dipping beds which form the syncline and was due to the depositional low rather than being fault related. The inflow of water into the area subsided soon after mining was completed.

2.3 ENGINEERING PROPERTIES OF MATERIAL

This section of the report presents the values of important material property parameters which characterize the various lithologic layers found in the Rainier Mesa tuff. The design of an underground cavern requires, at a minimum, the physical properties, the compressive and tensile strengths, and the elastic moduli of the media. The proposed site, located between the U12e.18 and U12e.07 drifts, has been partially characterized as a result of previous underground nuclear testing. In addition, a substantial amount of data is

available from other locations in the mesa but for the same lithologic units present at the proposed site. The various parameter values are presented for each of the pertinent lithologic layers and summarize laboratory measured properties for the proposed site, the U12e.14 area, and the entire mesa. Data for the proposed site was obtained from the UE12e#1, U12e.15 UG-2, and U12e.14 UG-10 drill holes identified previously. Similarly, the values for the U12e.14 area are based upon data from nine drill holes in the vicinity of the U12e.14 drift while the values for the entire mesa are obtained from drill hole data from the U12e, U12n, and U12t tunnel systems. The tabulated information includes the mean values, the standard deviation, and the number of data points used in the average.

2.3.1 Physical Properties

The as received (in-situ) density, dry bulk density, grain density, water content by weight, total porosity, saturation, gas-filled voids, and permanent compaction are presented in Tables 2-1 through 2-9 respectively. Data presented in these tables include mean property values, 1σ standard deviation, and the number of samples upon which the mean and standard deviation are based. The percentage of gas-filled voids in the core samples is as calculated from the densities and as measured by hydrostatic compression and uniaxial strain tests. It can be seen from these tables that the average parameter values are reasonably uniform throughout the mesa for each lithologic layer and that the values at the proposed site are in the range of the mean data from all areas in the mesa. It should be noted that the available core samples from tunnel beds 4F through 4H at the proposed site are limited and therefore any variation from the overall mesa property values may simply be due to the limited data base.

2.3.2 Core Index

The competancy of the rock formations is of considerable interest in the excavation of underground chambers. The engineering characteristics of the core samples per drilled interval is indicated by a "Core Index" value. The core index (CI) is a numerical representation of the joint frequency, core loss, and broken core of a sample and is determined by the following equation:

$$CI = \frac{(\text{ft broken core}) + (\text{ft core loss}) + (1/2 \text{ joints})}{(\text{ft drilled interval})} \times 100$$

An increase in the core index value is indicative of a corresponding increase in joint frequency, core loss, and amount of broken core; and thus a decrease in the competancy of the rock. A core index value below 50 generally represents competent rock formations. A second means of characterizing the competency of the rock is by means of the Schmidt hammer method in which rebound values below 20 generally indicate weak or incompetent rock (Reference 14). The CI and Schmidt hammer graphs for the U12e.15 UG-2, U12e.14 UG-10, and UE12e #1 drill holes are included in Plates 1 through 3 and indicate a high level of rock competency in the region of tunnel beds #4. There only appears to be a potential for incompetent rock formations where the U12e.14 UG-10 drill hole crosses a fault zone and as the U12e.15 UG-2 drill hole approaches the syncline (Figure 2-4).

2.3.3 Ultrasonic Velocity & Electrical Resistivity

Several methods have been successfully used to detect the presence of soft clayey zones or zones with high gas voids along the axis of a drill hole. The USGS has standardized two techniques, electrical resistivity and sonic velocity, which are useful to the siting of a large cavity.

The electrical resistivity technique is a diagnostic of broad zones in which clay is present in the rock. Such zones would, of course, be detrimental to the mining and stability of a large underground chamber. Empirical experience in Rainier Mesa indicates that clay zones are present where the resistivity is less than about 20 ohm-meters. The resistivity logs are included in Plates 1, 2 and 3 for the three drill holes in the vicinity of the the proposed site and indicate the absence of any large clay zones in the lithology sampled. It is of interest to compare the electrical resistivity logs in the two horizontal and one vertical drill hole and note the similar character in tunnel bed 4J which has been identified on all three logs.

Sonic Velocity is a technique utilized in Rainier Mesa tuff as an indication of the presence of zones with high gas voids. The ultrasonic longitudinal and shear wave velocities are presented in Tables 2-10 and 2-11 respectively. The average ultrasonic velocities in the area of the proposed site are comparable to other velocity data and do not suggest major anomalies

in the tuff. Average field sonic compressional velocities are on the order of 2590 m/sec (8500 ft/sec) while shear wave velocities are approximately 1250 m/sec (4100 ft/sec). The seismic survey results from the U12e.15 UG-2 drill hole are shown in Figure 2-12. These data can be obtained by means of the standard dynamite-geophone techniques or the newly developed shear-wave method. The latter technique is designed to obtain shear and compressional wave velocities, as well as amplitudes, in a dry borehole and appears to be a very valuable tool in assessing the strength and fracture condition of rock. The ratio of the field seismic compressional velocity to the laboratory sonic compressional velocity provides a good measure of the rock mass quality. In addition, sonic velocity techniques can be used to indicate the depth of rock fracturing and rock loosening around the surface of the opening which takes place during cavity excavation.

2.3.4 Elastic Moduli

The Young's moduli and Poisson's ratio, as scaled from the slopes of triaxial compression stress-strain curves, are presented in Table 2-12. The data are from the U12n tunnel complex (Mighty Epic, Diablo Hawk and Husky Ace) with the exception of several data points obtained for the U12e.14 event (Dido Queen). It appears that the U12n tuff, which is primarily from tunnel bed 3, produces a slightly higher Young's moduli and Poisson's ratio than tunnel bed 4K near the U12e.14 drift. An estimate of the elastic constants at overburden stress in the tunnel bed 4 of the proposed site are a Young's moduli of 20 to 30 kilobars (290 to 435 ksi) and a Poisson's ratio of 0.20 to 0.25. If further tests are conducted, these might prove conservative.

2.3.5 Compressive and Tensile Strengths

As in the case of the elastic constants, the majority of the triaxial compression test data are in the tunnel bed 3. However, a large number of uniaxial strain tests have been conducted in the tunnel beds 4 from which a failure surface can be estimated. The data from the subject area suggests that the lower portion of the shaded region in Figure 2-13 would be representative of the tuff compressive strengths. At an assumed overburden stress of about 70 bars (1015 psi), the compressive shear strength ($\tau = (\sigma_1 - \sigma_3)/2$)

would range from 0.1 to 0.15 kilobars (1450 to 2175 psi) with a mean at about 0.125 kilobars (1810 psi).

Unconfined tensile tests have been conducted on tunnel bed 3 material and the range is from 9 to 23 bars (130 to 335 psi). The tensile strength for tunnel bed 4 tuff would probably be on the order of 15 bars (215 psi).

2.4 IN-SITU STRESS IN RAINIER MESA

The in-situ state of stress has been determined by the U.S. Bureau of Mines overcore method (Reference 16) at nine locations throughout Rainier and Aqueduct Mesas. Results of these stress determinations indicate a generally characteristic pattern of stress orientation and magnitude within Rainier and Aqueduct mesas. Normally the maximum principal stress axes align to the northeast and are inclined within ±40° of horizontal. Conversely, the minimum principal stress components align to the northwest and are normally within a few degrees of horizontal. This pattern is very obvious when the principal stress axes are resolved into maximum and minimum horizontal stress components. Figure 2-14 is a plot of these horizontal secondary principal stress components for the nine mesa locations. Another characteristic of Rainier Mesa stresses is that the maximum horizontal stress magnitude is approximately equal to, or in most cases, slightly greater than the vertical stress magnitude.

For the nine locations in Rainier and Aqueduct Mesas the maximum stress magnitude ranges from a low of 6.6 MPa (960 psi) to a high of 11.7 MPa (1700 psi), with an average of 8.1 MPa (1150 psi). The minimum stress magnitudes range from a low of 1.4 MPa (200 psi) to a high of 5.7 MPa (830 psi) with an average of 3.2 MPa (465 psi). For six of the mesa locations the bearing of the maximum stress component averages North 36° East and is inclined within ± 40° of horizontal while at two others the maximum stress component is near vertical with the intermediate stress components bearing northeast. One location appears anomalous, giving a maximum stress direction approximately east-west. It should be noted that the bearing of the maximum and minimum stress components seems to roughly align with the topographic boundary of Rainier Mesa, as can be seen in Figure 2-14. Thus, orienation of the principal stress components is

considered to be dependent on location within the mesa. The vertical stress components determined at tunnel level at the nine locations average 6.3 MPa (915 psi) and range from a low of 5.6 MPa (810 psi) to a high of 7.4 MPa (1075 psi).

Stress conditions expected to exist between the U12e.18 and U12e.07 drifts can best be inferred from the stress determination conducted at the U12e.18 WP in 1974 (Reference 18). The U12e.18 WP is located approximately 220 m (720 ft) north-northwest of the hypothetical site. There is no reason to expect stress conditions at the proposed site to be significantly different than those determined at the U12e.18 WP. Table 2-13 lists the stresses determined at the U12e.18 WP and Figure 2-15 is a graphical representation of these stresses. These stress values are used for purposes of this preliminary feasibility evaluation.

The stresses determined at the U12e.18 WP differ slightly from the "average" stress pattern in Rainier Mesa in that the maximum stress bearing is more nearly north-south and plunges to the north at a relatively steep angle of 48°. This tendency of the maximum stress component to align more nearly north-south may be characteristic of the E-tunnel area. The magnitudes of the principal stresses are very representative of the Rainier Mesa averages.

Some mention of factors which could cause stresses in the study area to differ from the U12e.18 stresses should be made. The presence of geologic faults in the area could cause stress variations. Also, the U12e.18 WP was on the north limb of an east-trending shallow depositional syncline, while the proposed site is on the south limb of the syncline. Explosion-induced stress effects from the U12e.18 (Dining Car) event could be a remote source of possible stress variation. It is not expected, however, that any possible stress differences between the U12e.18 WP and the proposed site would be significant.

TABLE 2-1

AS-RECEIVED DENSITY, GM/CC

Rock Unit	Rainier Mesa	U12e.14 Area	Proposed Site
4K	1.91 ± 0.07 (64)*	1.91 ± 0.07 (64)	1.89 ± 0.08 (21)
4J	1.96 ± 0.08 (43)	1.96 ± 0.08 (40)	1.94 ± 0.07 (23)
4H	1.89 ± 0.06 (27)	1.88 ± 0.06 (20)	1.83 ± 0.04 (5)
4G	1.88 ± 0.06 (12)	---	1.81 (1)
4F	1.87 ± 0.07 (21)	---	1.83 ± 0.03 (2)
4E	1.92 ± 0.06 (10)	---	---
4CD	1.92 ± 0.08 (14)	---	---
4AB	1.93 ± 0.07 (15)	---	---
4	1.91 ± 0.08 (210)	---	---

* Mean value, standard deviation, and number of samples

TABLE 2-2

DRY DENSITY, GM/CC

Rock Unit	Rainier Mesa	U12e.14 Area	Proposed Site
4K	1.56 ± 0.10 (51)	1.56 ± 0.10 (51)	1.54 ± 0.13 (21)
4J	1.61 ± 0.13 (27)	1.59 ± 0.12 (24)	1.59 ± 0.09 (23)
4H	1.51 ± 0.09 (12)	1.48 ± 0.07 (10)	1.44 ± 0.03 (5)
4G	1.56 ± 0.11 (13)	---	1.45 (1)
4F	1.55 ± 0.12 (33)	---	1.44 ± 0.03 (2)
4E	1.58 ± 0.08 (10)	---	---
4CD	1.57 ± 0.13 (14)	---	---
4AB	1.59 ± 0.12 (15)	---	---
4	1.57 ± 0.11 (179)	---	---

TABLE 2-3

GRAIN DENSITY, GM/CC

Rock Unit	Rainier Mesa	U12e.14 Area	Proposed Site
4K	2.44 ± 0.09 (30)	2.44 ± 0.09 (30)	2.44 ± 0.04 (19)
4J	2.48 ± 0.05 (9)	2.50 ± 0.04 (6)	2.48 ± 0.06 (18)
4H	2.44 ± 0.01 (2)	---	2.43 ± 0.01 (5)
4G	2.41 ± 0.05 (13)	---	2.47 (1)
4F	2.40 ± 0.04 (33)	---	2.47 ± 0.03 (2)
4E	2.42 ± 0.06 (10)	---	---
4CD	2.40 ± 0.05 (11)	---	---
4AB	2.46 ± 0.05 (13)	---	---
4	2.42 ± 0.06 (125)	---	---

TABLE 2-4

WATER CONTENT, % BY WET WEIGHT

Rock Unit	Rainier Mesa	U12e.14 Area	Proposed Site
4K	18.4 ± 3.0 (51)	18.4 ± 3.0 (51)	18.9 ± 3.0 (21)
4J	18.3 ± 2.9 (27)	18.7 ± 2.6 (24)	18.3 ± 1.8 (23)
4H	20.3 ± 3.5 (14)	21.5 ± 2.7 (10)	21.4 ± 0.5 (5)
4G	19.6 ± 3.4 (10)	---	20.0 (1)
4F	19.2 ± 3.0 (21)	---	21.5 ± 0.7 (2)
4E	17.5 ± 1.7 (10)	---	---
4CD	18.5 ± 3.6 (14)	---	---
4AB	17.8 ± 3.3 (15)	---	---
4	18.5 ± 3.2 (166)	---	---

TABLE 2-5

POROSITY, % BY VOLUME

Rock Unit	Rainier Mesa	U12e.14 Area	Proposed Site
4K	35.3 ± 4.0 (30)	35.3 ± 4.0 (30)	37.0 ± 5.1 (19)
4J	32.3 ± 5.2 (9)	33.5 ± 5.1 (6)	35.5 ± 3.5 (18)
4H	33.8 ± 5.6 (4)	---	40.4 ± 1.1 (5)
4G	36.3 ± 4.5 (16)	---	41.0 (1)
4F	35.2 ± 5.1 (34)	---	42.0 ± 0.0 (2)
4E	35.2 ± 2.9 (10)	---	---
4CD	34.9 ± 6.0 (11)	---	---
4AB	35.3 ± 5.5 (13)	---	---
4	35.0 ± 4.8 (131)	---	---

TABLE 2-6

SATURATION, % OF VOIDS FILLED

Rock Unit	Rainier Mesa	U12e.14 Area	Proposed Site
4K	96.5 ± 7.7 (30)	96.5 ± 7.7 (30)	95.3 ± 4.5 (19)
4J	97.2 ± 3.4 (9)	97.0 ± 3.5 (6)	96.8 ± 4.1 (18)
4H	97.2 ± 3.2 (4)	---	95.4 ± 3.4 (5)
4G	96.7 ± 4.9 (10)	---	87.0 (1)
4F	96.3 ± 4.1 (20)	---	94.0 ± 0.0 (2)
4E	94.0 ± 6.1 (9)	---	---
4CD	98.1 ± 3.3 (9)	---	---
4AB	96.3 ± 2.5 (12)	---	---
4	96.0 ± 7.0 (107)	---	---

TABLE 2-7

CALCULATED AIR VOIDS, %

Rock Unit	Rainier Mesa	U12e.14 Area	Proposed Site
4K	1.4 ± 2.7 (30)	1.4 ± 2.7 (30)	1.7 ± 1.6 (19)
4J	0.9 ± 1.1 (8)	1.1 ± 1.2 (6)	1.1 ± 1.6 (18)
4H	1.0 ± 1.6 (3)	---	1.9 ± 1.4 (5)
4G	1.0 ± 1.1 (10)	---	5.3 (1)
4F	1.2 ± 1.3 (19)	---	2.0 ± 0.6 (2)
4E	1.7 ± 1.6 (8)	---	---
4CD	0.7 ± 1.2 (9)	---	---
4AB	1.4 ± 0.9 (12)	---	---
4	1.4 ± 2.1 (103)	---	---

TABLE 2-8

PERMANENT COMPACTION, % (HYD)

Rock Unit	Rainier Mesa	U12e.14 Area	Proposed Site
4K	1.4 ± 0.7 (32)	1.4 ± 0.7 (32)	---
4J	2.1 ± 1.9 (31)	2.1 ± 1.9 (31)	---
4H	2.6 ± 1.9 (20)	2.8 ± 2.0 (17)	---
4G	0.7 ± 0.3 (2)	---	---
4F	---	---	---
4E	0.8 (1)	---	---
4CD	1.4 ± 2.0 (4)	---	---
4AB	0.9 ± 0.3 (4)	---	---
4	1.8 ± 1.6 (94)	---	---

TABLE 2-9

PERMANENT COMPACTION, % (1-D)

Rock Unit	Rainier Mesa	U12e.14 Area	Proposed Site
4K	1.8 ± 1.5 (6)	1.8 ± 1.5 (6)	1.7 ± 0.9 (23)
4J	1.1 ± 1.0 (2)	1.1 ± 1.0 (2)	1.5 ± 0.6 (23)
4H	1.6 ± 2.0 (3)	1.6 ± 2.0 (3)	1.9 ± 1.1 (5)
4G	1.5 ± 0.3 (3)	---	3.1 (1)
4F	2.0 ± 0.7 (7)	---	1.7 ± 1.0 (2)
4E	0.8 ± 0.4 (4)	---	---
4CD	1.5 ± 1.0 (8)	---	---
4AB	2.0 ± 1.1 (10)	---	---
4	1.7 ± 1.1 (43)	---	---

TABLE 2-10

ULTRASONIC LONGITUDINAL VELOCITY, FT/SEC

Rock Unit	Rainier Mesa	U12e.14 Area	Proposed Site
4K	8640 ± 1100 (23)	8640 ± 1100 (23)	8890 ± 1130 (19)
4J	9580 ± 850 (4)	9580 ± 850 (4)	8830 ± 700 (13)
4H	---	---	8750 ± 700 (5)
4G	9550 ± 1180 (3)	---	7130 (1)
4F	9250 ± 2920 (11)	---	7640 ± 700 (2)
4E	9690 ± 1380 (7)	---	---
4CD	7980 ± 460 (4)	---	---
4AB	8640 ± 990 (11)	---	---
4	8920 ± 1570 (63)	---	---

TABLE 2-11

ULTRASONIC SHEAR VELOCITY, FT/SEC

Rock Unit	Rainier Mesa	U12e.14 Area	Proposed Site
4K	4840 ± 460 (5)	4840 ± 460 (5)	4400 ± 1000 (19)
4J	4730 (1)	---	4100 ± 600 (13)
4H	---	---	4440 ± 500 (5)
4G	4530 ± 980 (3)	---	3390 (1)
4F	5220 ± 1100 (9)	---	3550 ± 90 (2)
4E	4590 ± 1140 (6)	---	---
4CD	4170 (1)	---	---
4AB	4574 ± 540 (8)	---	---
4	4860 ± 980 (33)	---	---

TABLE 2-12

APPARENT ELASTIC CONSTANTS

	Area	E (kb) (average, standard deviation, and range)	ν (average, standard deviation, and range)
UNCONFINED			
	ME & DH	32* ± 23 (2-80)	.25 ± .11 (.05-.40)
	DQ	8*	.12
σ_3 = 34 bars (500 psi)	ME	58 ± 14 (44-75)	.23 ± .02 (.20-.27)
	DH	35 ± 27 (13-86)	.27 ± .05 (.17-.32)
σ_3 = 50 bars			
	ME	42 ± 38	.30 ± .05
σ_3 = 69 bars (1000 psi)	ME	60 ± 25 (37-95)	.24 ± .01 (.23-.25)
	DH	39 ± 23 (22-82)	.27 ± .05 (.20-.35)
σ_3 = 100 bars			
	ME	44 ± 19 (31-58)	.31 ± .09 (.24-.37)
σ_3 = 500 bars			
	DQ	19	.22
	MN		.30 ± .05
σ_3 = 4 kb			
	HA	50-70	.30-.35
	DQ	25-50	.27-.30

* Scaled from the linear portion of the curve (i.e. does not include "foot" on stress-strain plot).

TABLE 2-13. CALCULATED STATE OF STRESS AT THE U12e.18 WORKING POINT

Stress	Magnitude MPa (psi)	Standard Deviation MPa (psi)	Bearing	Inclination + degrees above horizontal - degrees below horizontal
		Principal stresses		
	(+, compression)			
S_1 (minimum)	+2.8 (406)	±.4 (58)	N. 75° W.	+12
S_2 (maximum)	+6.9 (1001)	±.4 (58)	N. 4° E.	-40
S_3 (intermediate)	+6.0 (870)	±.4 (58)	N. 28° E.	+48
	Normal stress components in X, Y, Z (east, north, vertical) coordinate system			
	(+, compression)			
σ_x	3.1 (450)	±.4 (58)	East	Horizontal
σ_y	6.3 (914)	±.4 (58)	North	Horizontal
σ_z	6.2 (900)	±.3 (44)	- -	Vertical
	Shear stress components in X, Y, Z coordinate system			
τ_{xy}	+.8 (116)	±.3 (44)	- -	- -
τ_{yz}	-.7 (102)	±.3 (44)	- -	- -
τ_{zx}	+.6 (87)	±.3 (44)	- -	- -

Positive or negative sign on shear stress magnitude indicates direction of shear stress with respect to X, Y, Z coordinate system.

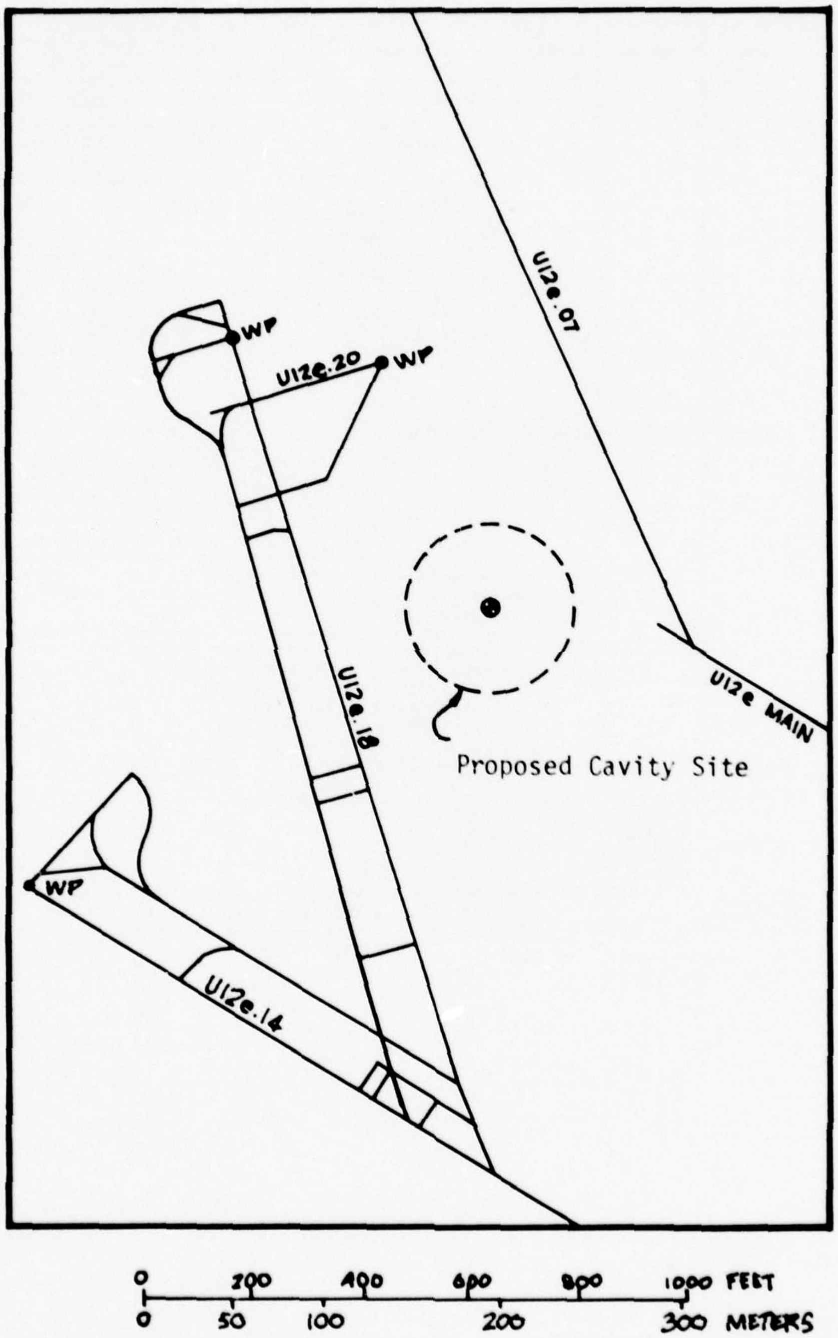


FIGURE 2-1. TUNNEL LEVEL LOCATION OF HYPOTHETICAL SITE IN E TUNNEL

Era	System	Series	Formation	Member or unit and symbol
CENOZOIC	Tertiary	Miocene	Timber Mountain Tuff	Rainier Mesa Member Tmr
			Paintbrush Tuff	Tiva Canyon Member Tpc Tp
			Stockade Wash Tuff Tsw	
			Bedded and ash-flow tuffs of Area 20 Trab	
			Bedded tuff of Dead Horse Flat Tdhb	
			Belted Range Tuff	Grouse Canyon Member Tbg
			Tunnel beds	Unit 5 Tt5
				Unit 4 Tt4 Subunits AB, CD, E, F, G, H, J, K[1]
				Unit 3 Tt3 Subunits A, BC, D[2]
			Belted Range Tuff	Tub Spring Member Tbt
			Tunnel bed	Unit 2 Tt2
			Crater Flat Tuff Tcf	
			Tunnel bed	Unit 1 Tt1
			Older tuffs (Redrock Valley Tuff, Fraction Tuff, and others) Tot	
MESOZOIC	Cretaceous		Quartz monzonite of Gold Meadows stock Kq	
PALEOZOIC	Devonian Silurian Ordovician Cambrian		Paleozoic rocks Pz	
PRECAMBRIAN			Precambrian rocks p€	

[1]K is the youngest.
[2]D is the youngest.

FIGURE 2-2. GENERAL STRATIGRAPHY OF RAINIER MESA

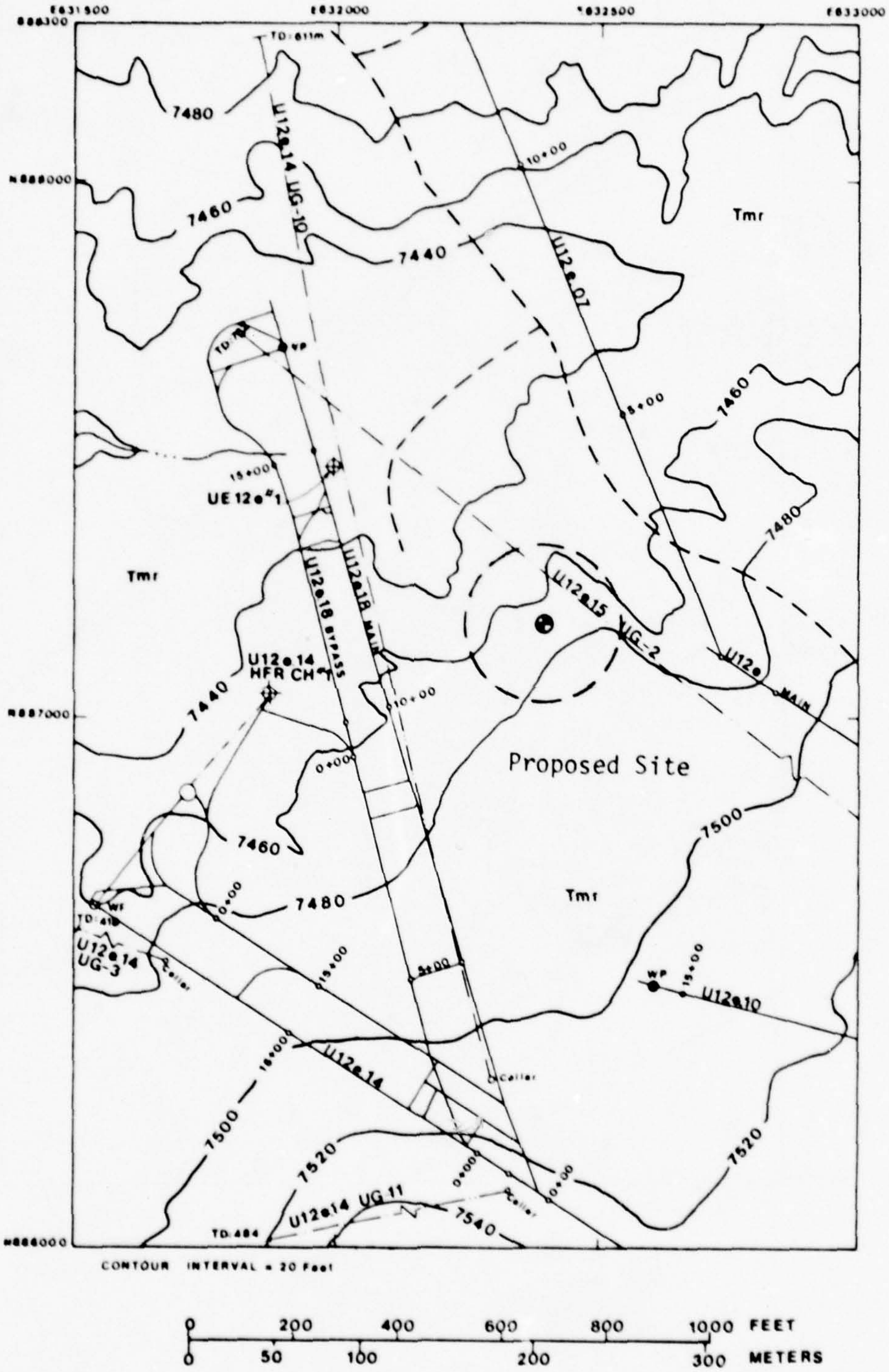


FIGURE 2-3. SURFACE CONTOUR LEVEL OVER THE U12e.18 TUNNEL

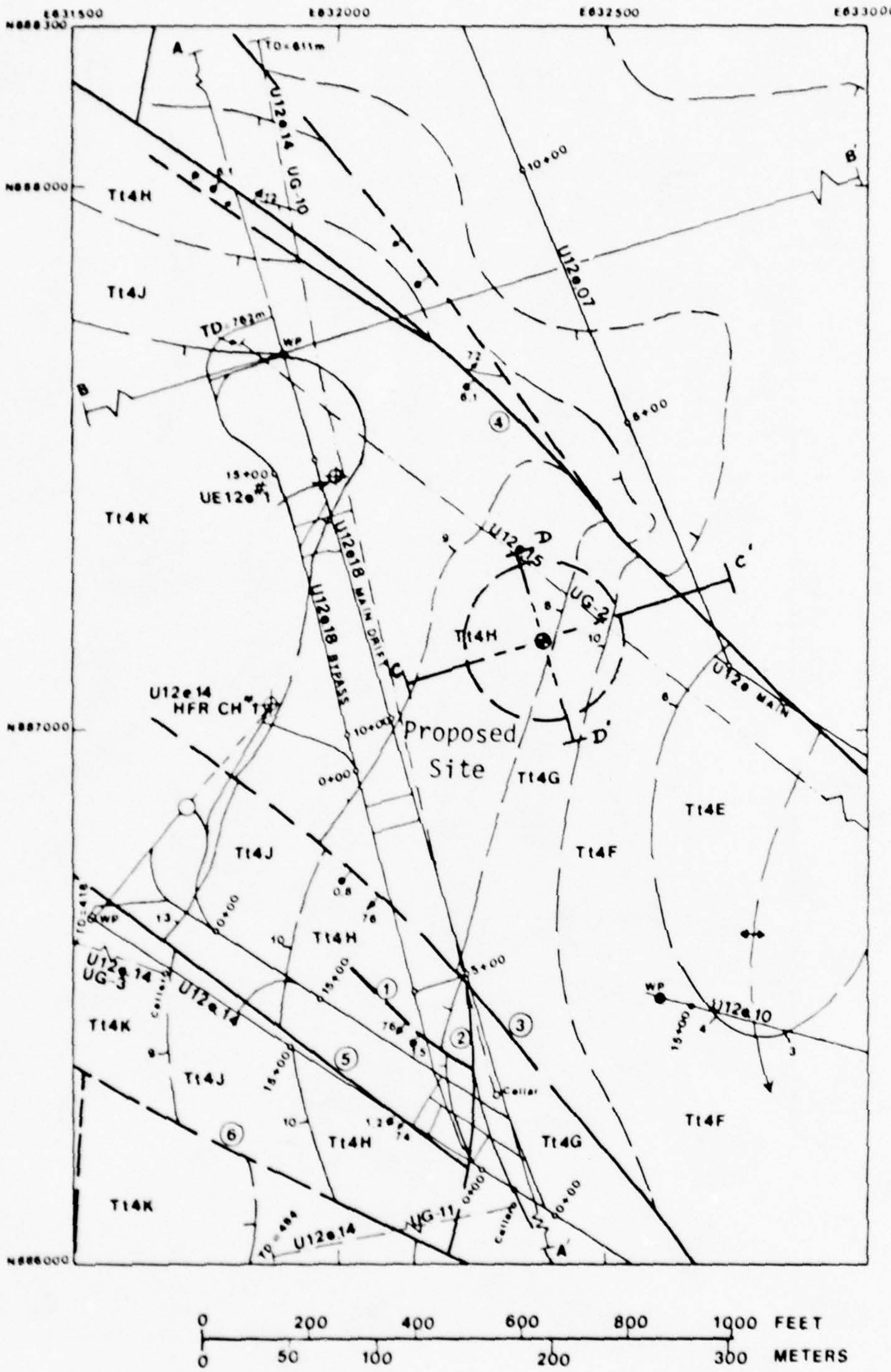


FIGURE 2-4. TUNNEL-LEVEL GEOLOGY OF THE U12e.18 AREA

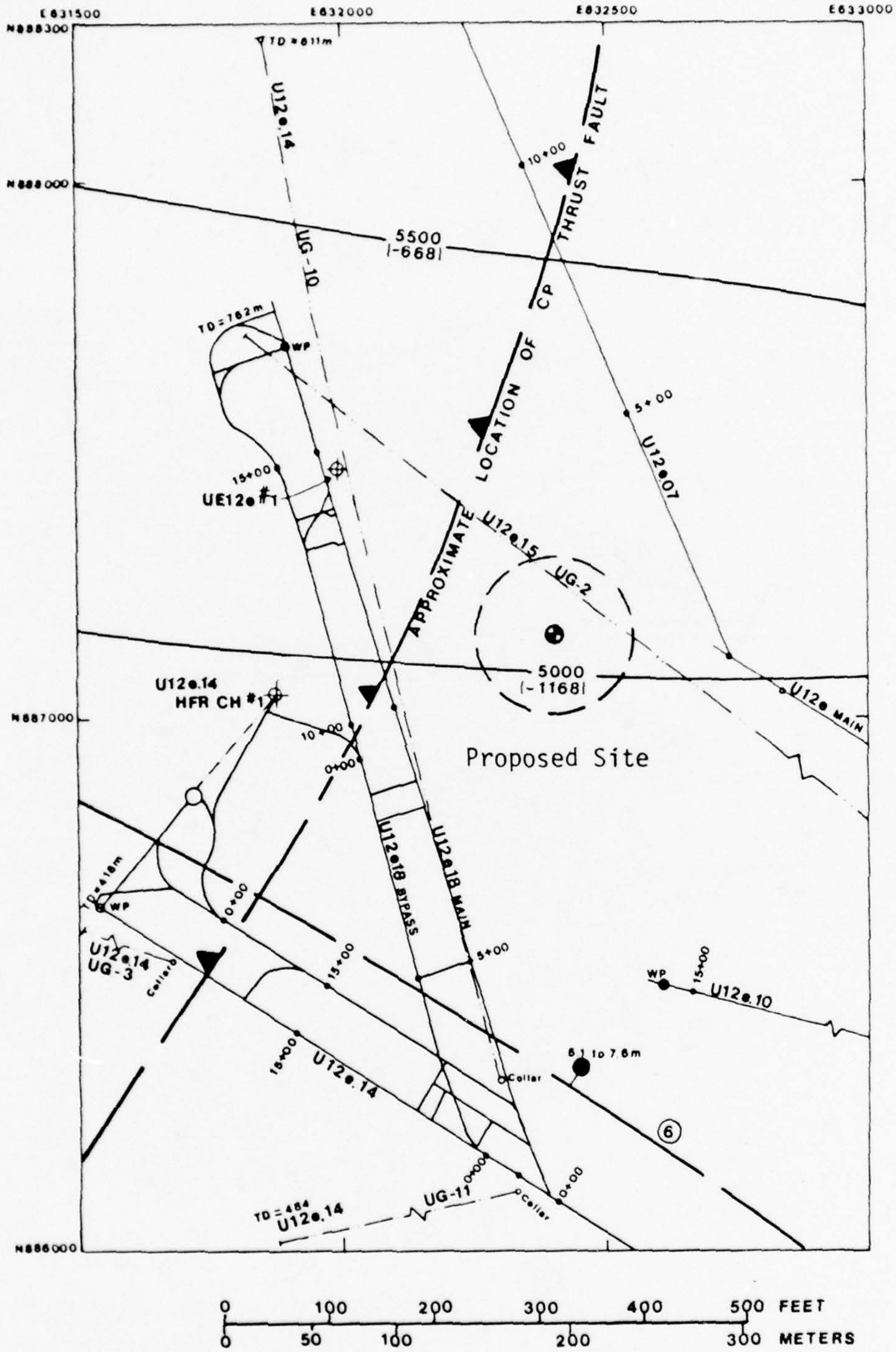


FIGURE 2-5. PRE-TERTIARY SURFACE STRUCTURE MAP OF THE U12e.18 AREA

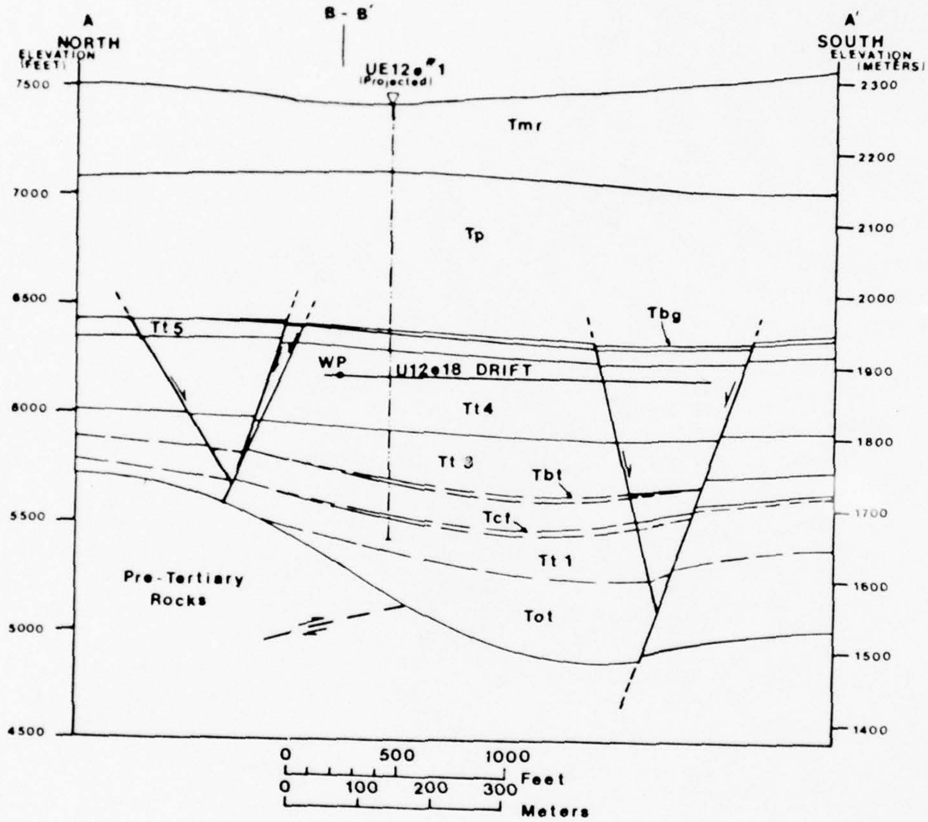


FIGURE 2-6. CROSS-SECTION ALONG U12e.18 DRIFT COLLAR AND WORKING POINT (Section A-A' of Figure 2-4)

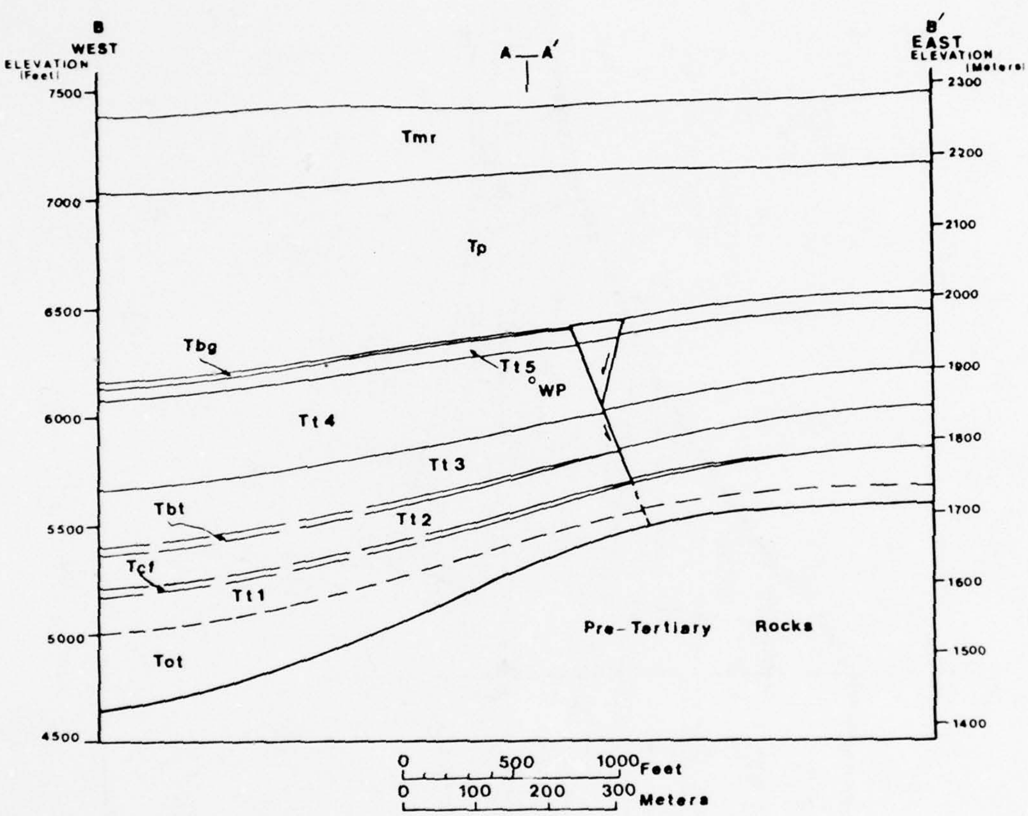


FIGURE 2-7. CROSS-SECTION PERPENDICULAR TO U12e.18 DRIFT AT THE WORKING POINT (Section B-B' of Figure 2-4)

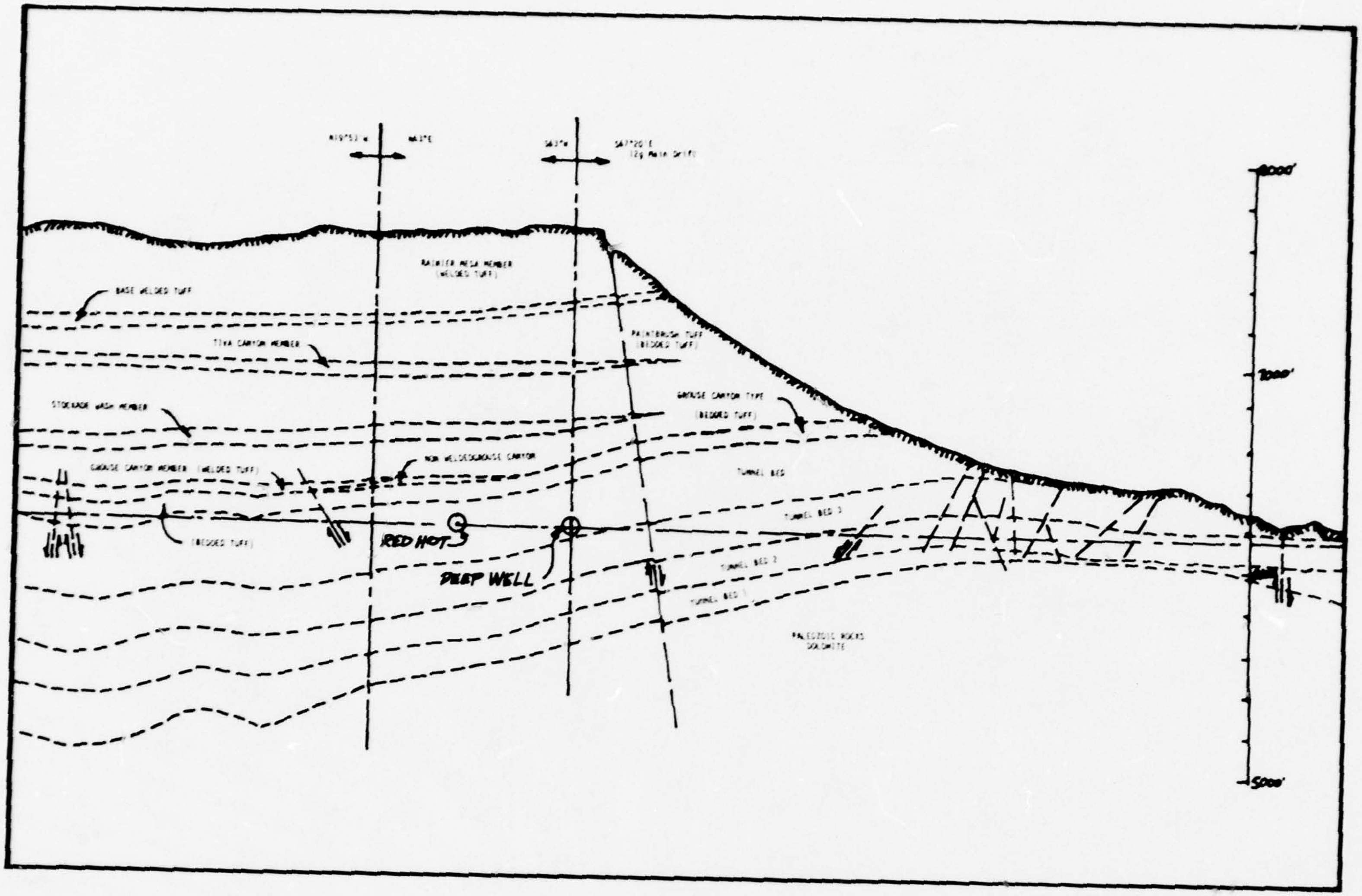


FIGURE 2-8. GENERAL CROSS-SECTION OF G TUNNEL AREA

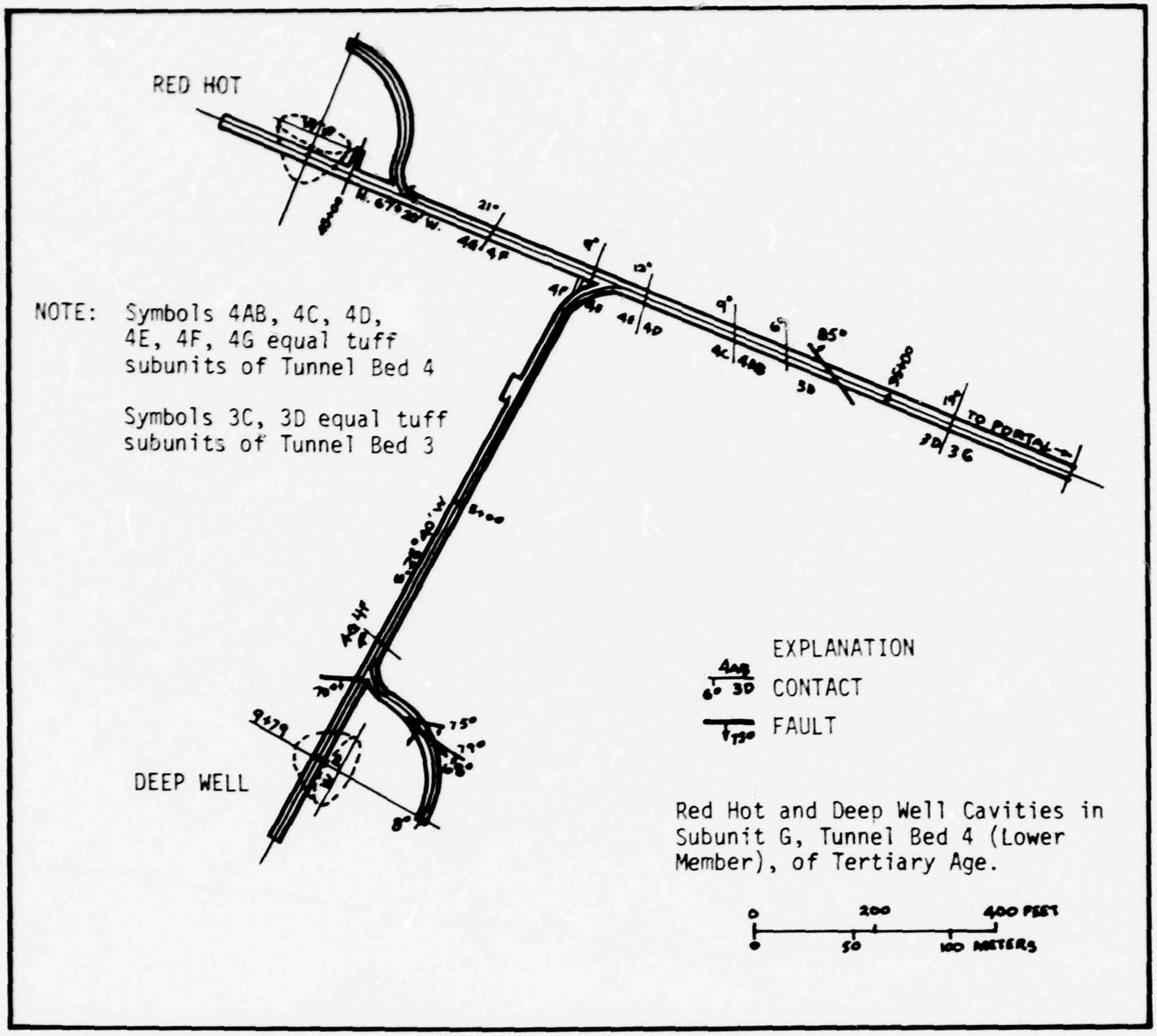


FIGURE 2-9. GEOLOGY IN THE VICINITY OF THE RED HOT AND DEEP WELL CAVITIES

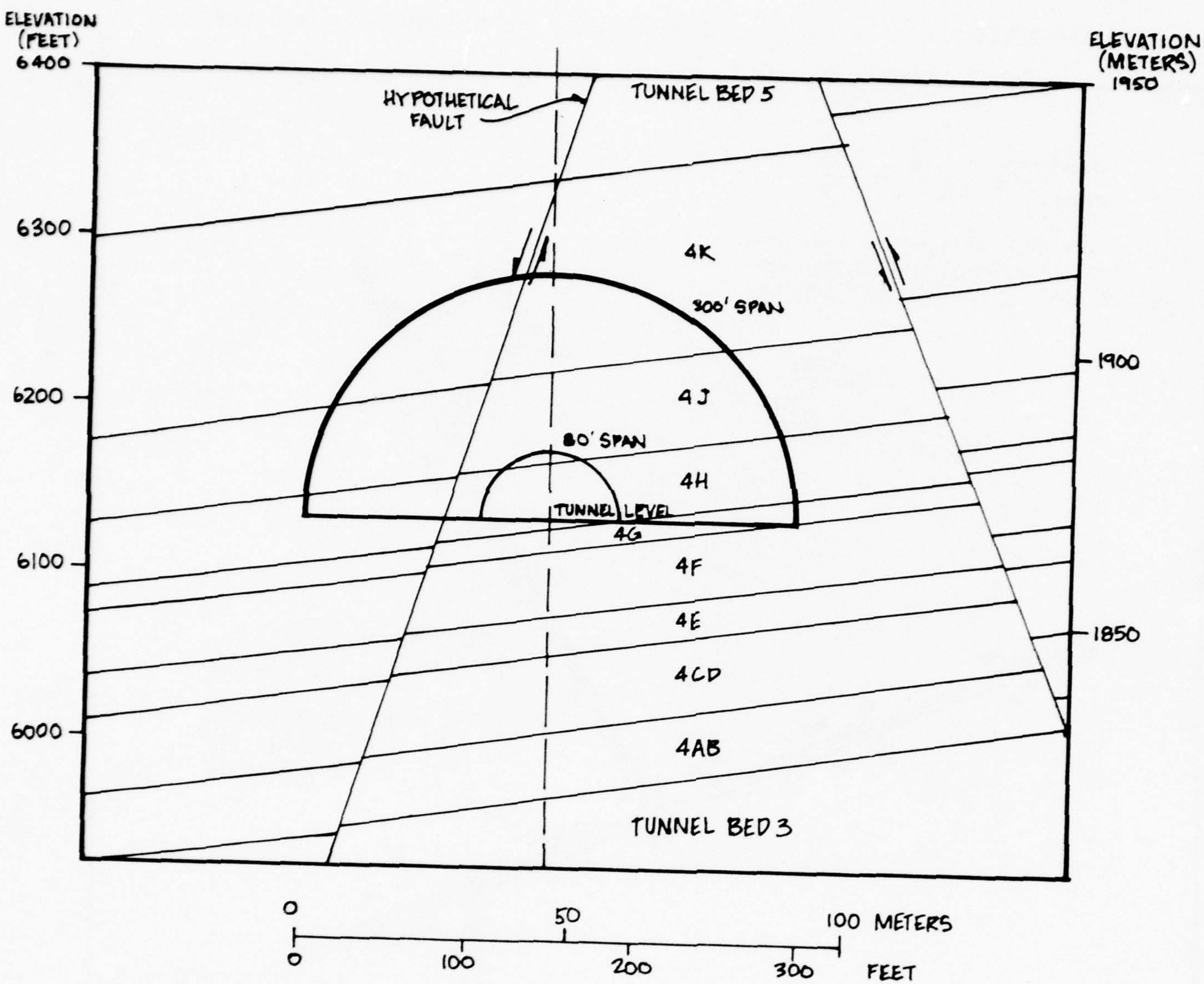


FIGURE 2-10. CROSS-SECTION THROUGH TYPICAL TUNNEL BED 4 SITE
(Section C-C' of Figure 2-4)

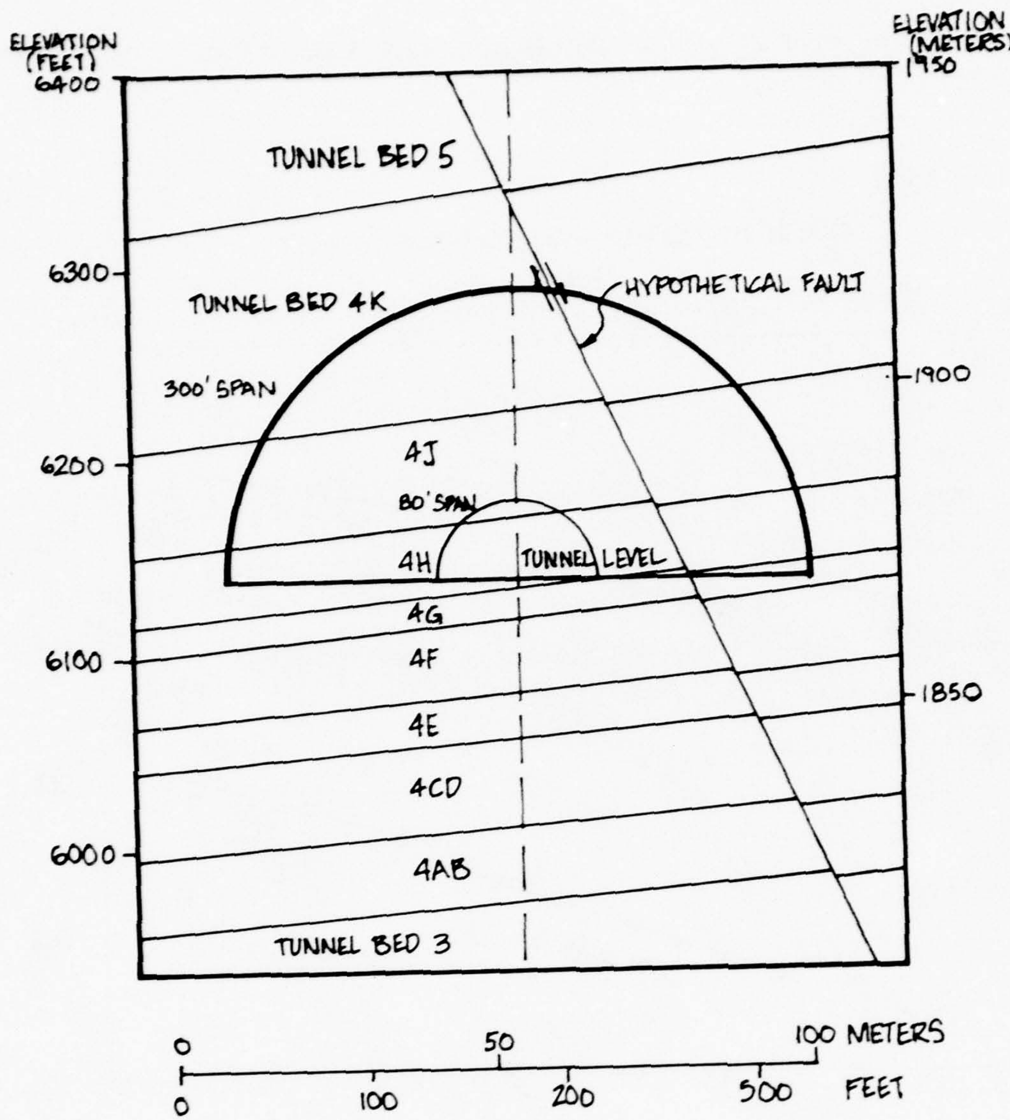


FIGURE 2-11. CROSS-SECTION THROUGH TYPICAL TUNNEL BED 4 SITE
(Section D-D' of Figure 2-4)

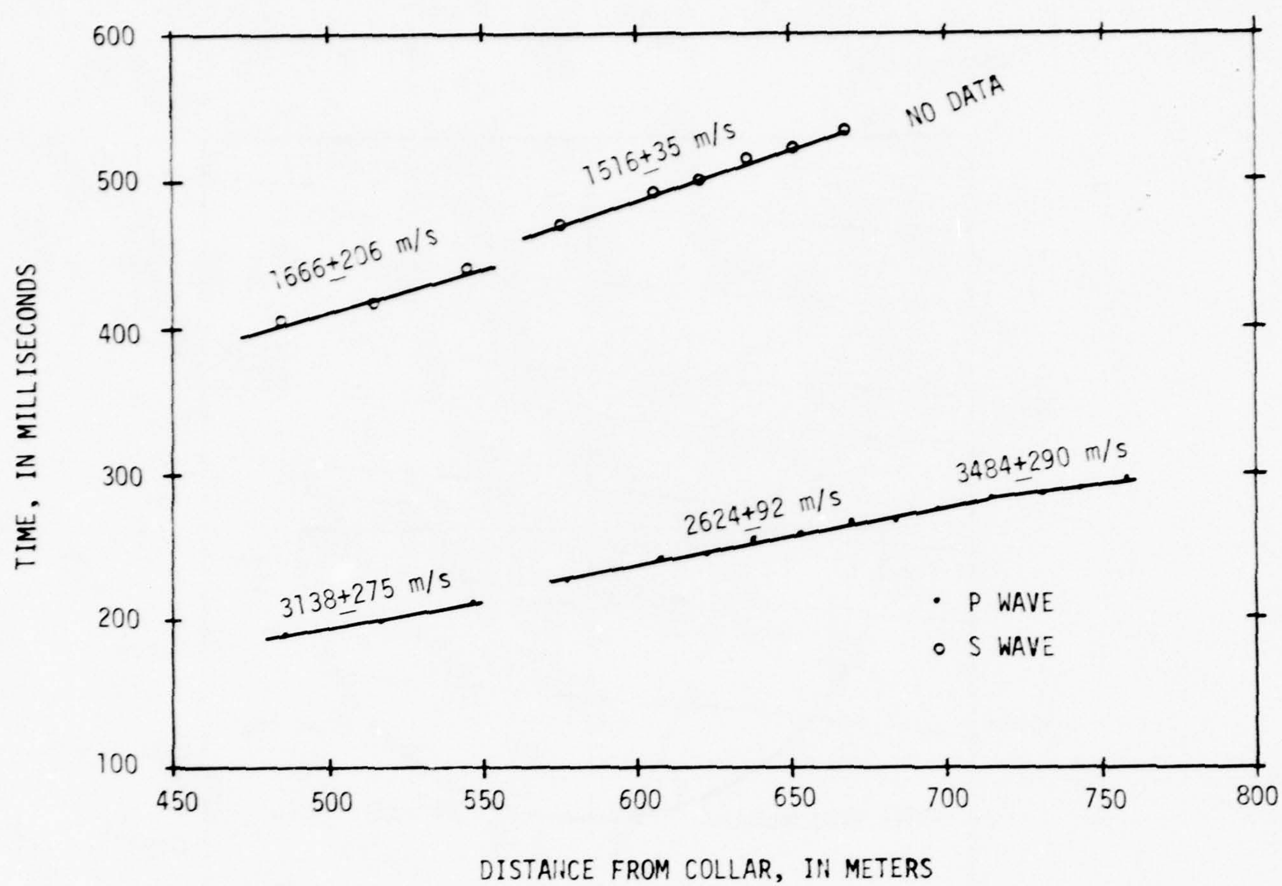


FIGURE 2-12. TIME-DISTANCE DATA FROM SEISMIC SURVEY OF U12e.15 UG-2 HORIZONTAL DRILL HOLE

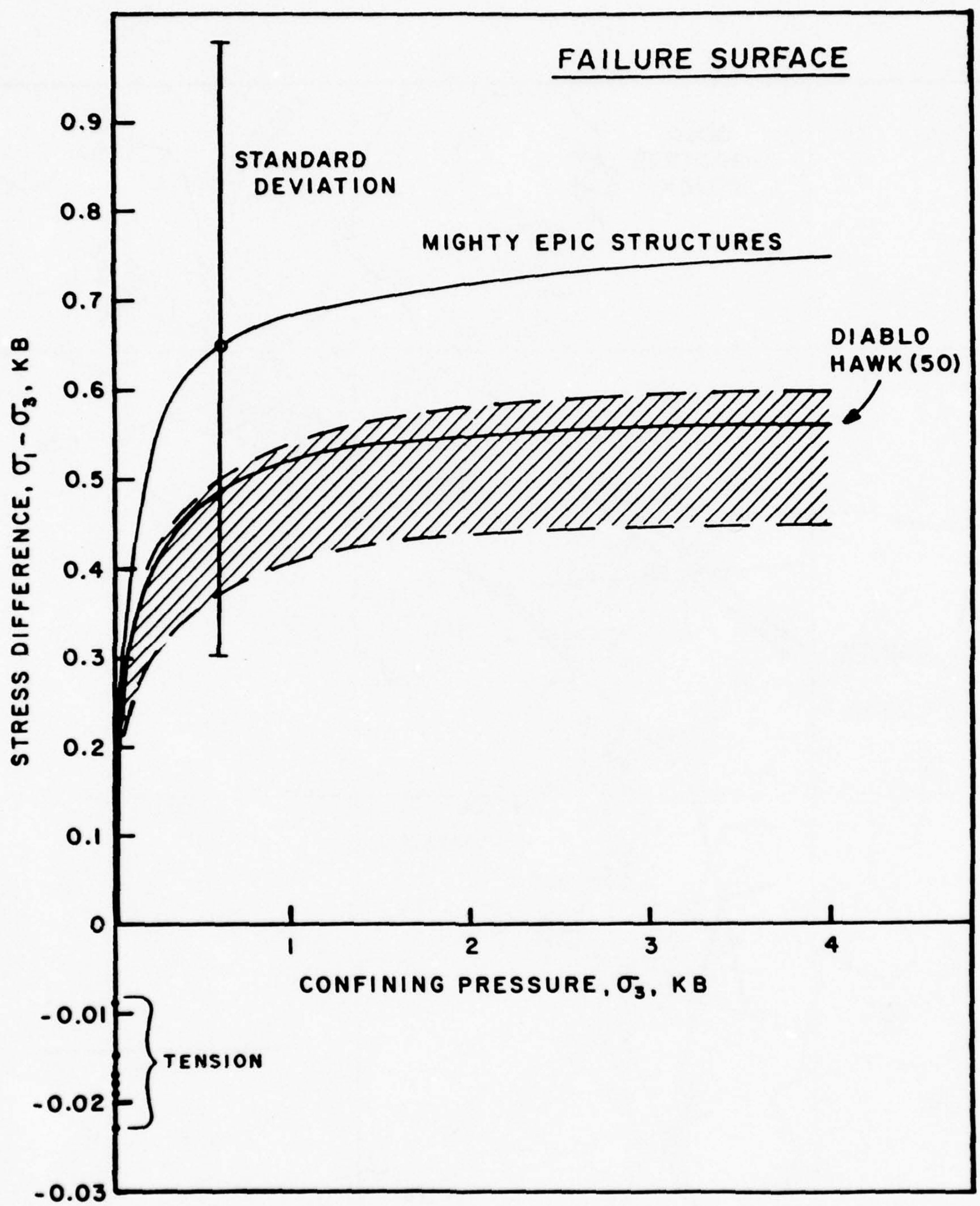


FIGURE 2-13. COMPRESSIVE AND TENSILE STRENGTHS OF TUFF

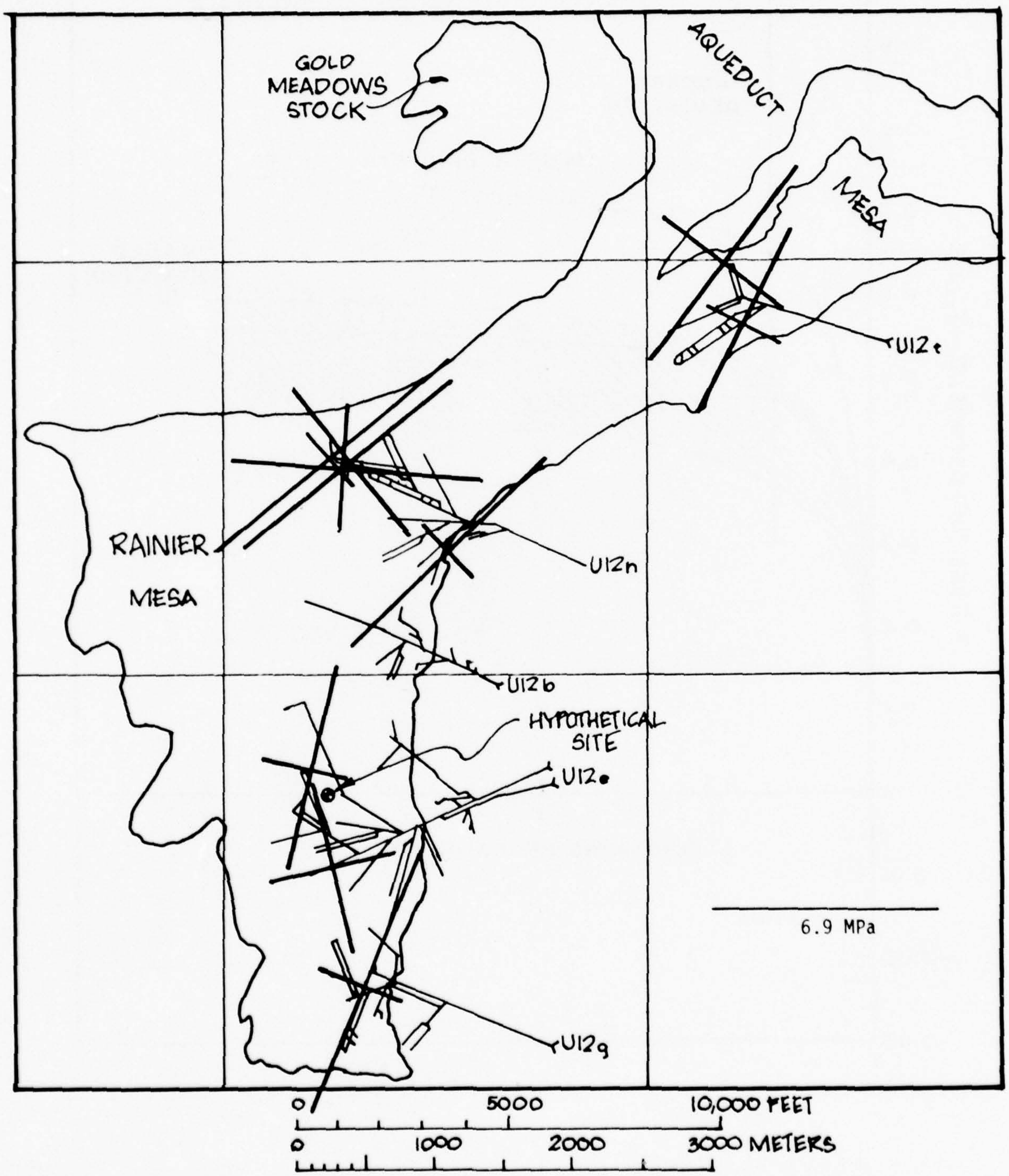
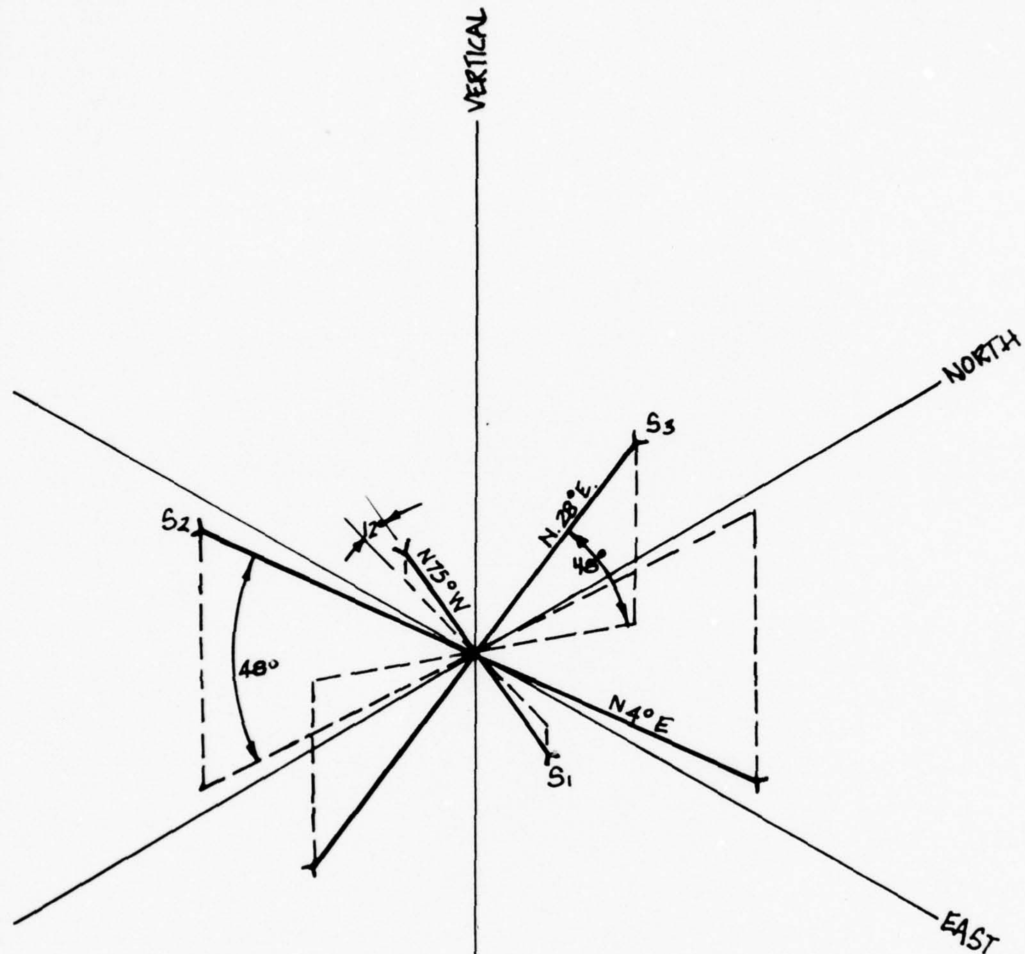


FIGURE 2-14. HORIZONTAL PRINCIPAL STRESS COMPONENTS IN RAINIER AND AQUEDUCT MESAS



S_1 = 2.8 MPa (406 psi)
S_2 = 6.9 MPa (1000 psi)
S_3 = 6.0 MPa (870 psi)

FIGURE 2-15. PRINCIPAL STRESSES AT U12e.18 (DINING CAR) WORKING POINT

3. PREVIOUS LARGE CAVITY EXPERIENCE

This section describes the experience gained from previous large cavity excavations in rock similar to that found at NTS. The size of various openings is documented together with a discussion of problems encountered, observed displacement and rock behavior, and influence of faulting, near-vertical joints, and bedding planes. The guidance and confidence that this past experience provides related to the excavation of even larger cavities is also discussed.

3.1 PAST EXPERIENCE WITH LARGE CAVERNS IN WEAK ROCK

Much of the experience with large permanent caverns in rock has been associated with underground hydroelectric power stations. The machine halls of these facilities usually have near-vertical walls on the order of 24 to 46 m (80 to 150 ft) high, are between 91 and 182 m (300 and 600 ft) in length, and have widths in the range of 15 to 30 m (50 to 100 ft). Hoek (Reference 19) documents the dimensions of most of the major cavern excavations and none are listed with widths greater than 33.5 m (110 ft). Consideration has been given to the construction of underground nuclear power stations with caverns up to 61 m (200 ft) in diameter, but none have been constructed to date.

Much of the experience gained from these past excavations is related to rock support requirements which include parameters such as support pressure, rockbolt length, and rockbolt density. These are necessarily dependent upon the specific geologic site conditions with the orientation of and strength along major discontinuities in the rock mass significantly affecting the support requirements. A summary of the rock support and rock movements of some of the well-documented large caverns is presented by Cording (Reference 12). Rock bolts or tendons provided initial, and in some cases permanent, support for the cavern roofs.

Many of the cavities described in the above references are located in good quality rock of high intact strength, since construction of large caverns for hydroelectric projects are usually considered only if the rock conditions are favorable. Table 3-1, and the following paragraphs, summarize the size, support, and rock movement of several more pertinent excavations, where large caverns were constructed in bedded deposits such as shales, mudstones, or tuff.

The Waldeck II (West Germany) and Hongrin (Switzerland) Powerhouses were both located in bedded deposits which exhibited unconfined compressive strengths more than 5 times that for the Rainier Mesa tuff. Both were approximately 30.5 m (100 ft) wide and the rock support for both consisted of long prestressed tendons, intermediate bolts, and shotcrete. The Hongrin chamber was supported by excavating perimeter drifts from bottom to top, placing support at the walls, and then removing the core after the walls and arch were supported. Additional 20 m (65 ft) long tendons were installed in the poor rock zones to provide increased support. Maximum rock movements of 2 cm (.8 in) were noted in the crown of the cavity. The Waldeck II Powerhouse was excavated by means of a top down-benching procedure, placing tendons at the wall as it was exposed during excavation. A maximum rock movement of 3.8 cm (1.5 in) occurred in a fault zone located in the crown of the cavern.

The Portage Mountain (British Columbia, Canada) Powerhouse was constructed in sandstones, siltstones, shales, and coal which exhibited unconfined compressive strengths 8 times that of the Rainier Mesa tuff. The cavern was supported by means of mechanically anchored rockbolts. Separation along bedding plane partings developed in the roof to a distance greater than 6 m (20 ft) above the crown, resulting in over 12.7 cm (5 in) of settlement of the roof. The magnitude of the rock movements may have been due to inadequate rock bolt support, possibly slippage of bolt anchors prior to grouting of the bolts.

The Drakensberg (South Africa) and Poatina (Tasmania) projects, both located in bedded siltstones, shales and sandstones, showed the effect of high horizontal stresses. In both cases the ratio of the unconfined com-

pressive strength to the maximum free field stress, q_u/σ_h, was approximately 2 to 3, being low enough to result in the formation of new fractures in the rock immediately around the openings. Small offsets took place along bedding plane partings near the haunch in both chambers. In the Drakensberg chamber, there was evidence of buckling of thin slabs due to the high stresses.

The Coo (Belgium) Pumped Storage Station was constructed in rocks of relatively high intact strength, but in an area that was heavily faulted and fractured. As a result, the concrete arch was placed after small excavations were opened, and large movements on the order of 12.7 cm (5 in) developed on the sidewalls. Although details of the construction procedure are not available, it is apparent that the procedures were not fully effective in limiting the rock movements. The quality of the rock was much lower and the faulting and fracturing much more extensive than would be encountered in a cavern in tuff in Rainier Mesa.

The Dehradun (India) Powerplant was constructed in slates and thinly laminated limestones. The rock exhibited surface slabbing due to the stress conditions. Joints and slip planes formed rock wedges in the sidewall which required support. Unlike any of the previous caverns discussed, the Dehradun Powerplant included exterior galleries which were mined outside the cavern walls prior to the excavation of the main chamber. As the main cavern was benched down in 3 m (10 ft) stages, pre-drilled anchor holes were exposed. The anchors were then fitted with baseplates, prestressed, and subsequently fully grouted between the wall and the gallery. This cavern is of particular interest since it bears similarity to the excavation and support methods proposed in this report for chambers exceeding 46 m (150 ft) diameter.

The caverns having the greatest similarity to the proposed caverns evaluated in this study are Caverns I (Red Hot) and II (Deep Well) which were also constructed in Rainier Mesa tuff. These cavities were excavated in 1965 in the "G" tunnel complex at NTS. It should be noted that significant differances exist between the Rainier Mesa tuff and the rock described for the other caverns. Most of the other cavities were constructed in rock of higher

strength than the tuff. However, the tuff is of excellent rock mass quality and has a lower frequency of major joints, faults and bedding plane weaknesses than the rock in which most of the other caverns listed in Table 3-1 were constructed.

The Red Hot and Deep Well cavities were 36.6 m (120 ft) diameter hemispheres with their flat base inclined at an angle of 72° to the horizontal. They were excavated using drill and blast procedures, beginning at the top of the chamber and proceding downward in vertical increments of approximately 1.8 m (6 ft). The exposed surface of the chamber was immediately supported with tensioned rockbolts, anchored with a cement grout. A thin layer of gunite was later applied to minimize drying and cracking of the tuff at the cavern surface. At the time the caverns were constructed, there was concern on the part of a few that the caverns could not be built because of the stress-induced fracturing that was expected to develop on the cavern walls. Careful construction observations, using extensometers, provided the means for evaluating the stability throughout construction and enabled the caverns to be built without an extremely conservative support system, even though some slabbing did develop.

New fractures developed around Cavities I and II to a depth of up to 2.4 m (8 ft) due to the low rock strength and relatively high in-situ stresses ($q_u/\sigma_v = 1.5$). The most prominent loosening of slabs formed by the new fractures developed to a depth of approximately .9 m (3 ft). Early placement of support helped limit the loosening, although it could not prevent new fractures from forming. As long as the support held these slabs in place, stability and safety could be maintained.

The rockbolt support on the face of Cavity II was initially light, in order to accommodate cavern user needs. It consisted of 7.3 m (24 ft) long rockbolts on a 1.8 x 1.8 m (6 by 6 ft) spacing, with a support pressure of 34 KPa (5 psi). Details of the rock supporting system for Cavity II are shown in Table 3-2.

Large movements developed on the plane face of Cavity II during excavation as a result of vertical joints and a bedding plane weakness that intersected to form an unstable rock wedge behind the face. New rock fracturing developed at one side of the wedge and contributed to its progressive instability. Additional rock bolts were placed on a .9 x .9 m (3 x 3 ft) pattern on the plane face of Cavity II, and the face was stabilized at a support pressure of 138 KPa (20 psi).

The plane face of Cavity I was in the same strength rock but was not intersected by major continuous, near vertical joints as was the Cavity II face. The initial 1.8 x 1.8 m (6 x 6 ft) rock bolt spacing was adequate in Cavity I, and no large movements developed. It is apparent that the difference in behavior between Cavity I and Cavity II was a result of the rock jointing behind the plane face of Cavity II. However, it is quite probable that the joint and bedding weaknesses in Cavity II would not have resulted in large movement of rock wedges had the rock been of higher intact strength. New fractures formed as the wedge moved, contributing to the unstable condition.

3.2 ROCK BEHAVIOR IN LARGE CAVERNS

From the documented behavior of these previously excavated caverns, particularly Cavity I and Cavity II mined in Rainier Mesa, the behavior of the tuff due to the presence of the opening and the effects of intersecting vertical joints and bedding plane weaknesses can be anticipated.

3.2.1 New Fractures

Because the stress concentrations around the opening will exceed the unconfined compressive strength of the rock, new fractures will develop within approximately 1.5 m (5 ft) of the cavern wall. The fractures will tend to form parallel to the surface of the cavern as excavation is carried out as shown in Figure 3-1. Further excavation downward will result in exposure of the previously formed fractures in the wall of the excavation. Small lateral offsets on the order of 3.1 to 12.7 mm (1/8 to 1/2 in) will develop along some of these fractures. By proper excavation and support, the loosening of slabs formed by these fractures can be minimized and the slabs can be supported.

New fractures may also combine with the vertical faults and bedding planes to form large wedges of rock requiring long, high capacity rockbolts or tendons for support.

3.2.2 Potential Rock Failure Modes

Possible modes of loosening and failure of the rock surrounding the cavern are illustrated in Figure 3-2 and are described as follows:

1. Loosening along bedding plane partings in the roof and the developing of slabs in thin bedded zones (Figure 3-2a). Timely rockbolt support and shotcreting will limit loosening, although some spalling may take place between rockbolts, particularly in zones of brittle tuff.

2. Loosening, drying, fallout or overbreak of slabs formed by the new fractures (Figure 3-2b). Shotcrete, in addition to rockbolts, will prevent the deterioration of the rock slabs.

3. Local loosening of shallow slabs where vertical joints or faults are parallel and adjacent to the side walls (Figure 3-2c).

4. Large wedges formed by combinations of vertical joints and faults, bedding plane weaknesses, and new fractures (Figure 3-2d). The cavern should be sited so that major N-S and E-W joints and faults do not intersect in the vicinity of the cavern. However, at least one near-vertical fault zone should be expected in the vicinity of the cavern, and the design rock anchor pattern must be adequate to support wedges formed by such a feature. The high-capacity tendons or rockbolts must be long enough to extend through these wedges and have enough capacity to hold them in place. It is ex-

pected that the critical wedges formed by the major joints and bedding plane weaknesses will not extend further than 0.3B from the cavern wall, where B is the cavern diameter.

5. Buckling and heave along bedding plane surfaces in the floor (Figure 3-2e). Rockbolts or tendons placed on the sidewall prior to excavation will prevent this phenomenon from causing instability of the wall. Bolting in the vicinity of the permanent floor may be required as the bottom of the cavern is approached.

3.3 ROCK SUPPORT IN LARGE CAVERNS

In order to maintain cavern stability and to prevent the above failure modes from developing, general rock support requirements have been established based upon a significant volume of information which has been gained from the documentation of previously mined caverns. Figures 3-3 and 3-4 respectively depict, as a function of cavern widths, the applied support pressure and the rockbolt lengths used to support the crown of various caverns. Rockbolt lengths in the arch of the cavities typically ranged from 1/4 to 1/3 the cavern width, although lengths up to 1/2 the cavern width were used for some of the high capacity tendon installations. The rockbolt spacing was generally established such that the bolts were capable of supporting rock loads equivalent to between $0.15\gamma B$ and $0.30\gamma B$, where B is the cavern width and γ the unit material weight. In other words, in a 30.5 m (100 ft) wide cavern, the load that the bolts were capable of supporting was equivalent to a height of rock extending between 4.6 and 9.1 m (15 and 30 ft) above the cavern roof. This is seen graphically in Figure 3-4.

Table 3-2 shows the bolt load, spacing and length and the support pressure used to support the tuff at various locations within the Red Hot and Deep Well cavities excavated in Rainier Mesa at NTS. Also shown is the increased support required to stabilize the plane face of Cavity II (Deep

Well). It should be expected that the excavation of a large cavity at the proposed location will face similar problems to those encountered in the mining of Cavity I (Red Hot) and Cavity II (Deep Well). It should also be expected that additional local support will be required in regions where the intersection of vertical joints and bedding weakness planes form unstable rock wedges.

3.4 CONCLUSIONS DRAWN FROM PAST EXPERIENCE

Several conclusions can be drawn from the case histories that have applicability to the feasibility and basic design criteria for a large chamber in tuff.

Underground machine halls are typically long structures, as opposed to the essentially equidimensional hemispherical chamber contemplated for the tuff. A machine hall having vertical walls of greater height and length than its 33.5 m (110 ft) width could be considered equivalent in size to a hemispherical opening having a diameter of approximately 45.7 to 54.9 m (150 to 180 ft). Thus, the construction of a 45.7 to 54.9 m (150 ft to 180 ft) diameter chamber would not be a significant departure from the current state-of-the-art.

The experience with the construction of two 36.6 m (120 ft) diameter caverns in the Rainier Mesa tuff over 12 years ago is also an indication that a 45.7 to 54.9 m (150 to 180 ft) diameter cavern is not a significant departure from previous experience. This is particularly true when one considers the improved excavation methods and support systems presently available and the substantial experience that was gained from the construction of the two caverns in tuff.

As discussed in this and following sections, it is technically feasible to construct hemispherical caverns of the 24.4 to 91.4 m (80 to 300 ft) size range studied in this report. Cavern diameters in excess of 45.7 m (150 ft) might be considered to be beyond current practice, but are not beyond the capabilities of the current-state-of-the-art. In designing a cavern 48.8 to 91.4 m (160 to 300 ft) in diameter, support pressures and rockbolt or

tendon lengths must be suitably scaled with the size of the opening in order to support the deeper rock wedges that can form around a larger opening. The primary geologic factors influencing the size of the critical rock wedges that must be supported in the tuff are the joints and bedding plane weaknesses and the stress-induced fractures that form during excavation.

The coordination and timing of support installation with respect to the excavation sequence has a major influence on rock movements and the stability of a cavern. In many caverns in excellent quality rock of high strength, the caverns can be adequately supported by placing internal supports after an excavation increment in a top-down benching sequence. However, early support, or pre-support of the rock mass using grouted rock bolts or grouted dowels placed from headings or from small drifts prior to excavation has provided a means of limiting movements and controlling the stability of a cavern or tunnel on may projects where ground conditions were difficult.

The conclusion that caverns up to 91.4 m (300 ft) in diameter are feasible in the tuff, is based upon the use of internal perimeter drifts and external annular galleries to limit and control rock displacements (Figure 3-5). From these development excavations, rock bolts and tendons are placed to pre-support the rock surrounding the cavern prior to general excavation of the cavern. Loosening that would otherwise develop along bedding plane partings and stress-induced fractures behind the wall and in the floor is minimized, and the type of movement that developed on the plane face of the Deep Well Cavity, described in Section 3.1, will be stopped before it begins to cause major stability problems.

The external galleries will also provide an accessible location for observing tendon loads and rock movements throughout excavation, and for installing additional support at any time during excavation if the measurements indicate the need for added support. Experience to date with caverns up to 36.6 m (120 ft) in span indicates that proper instrumentation of the cavity during excavation will provide advanced information concerning potential rock mass instability and that such instability can be remedied by the installation of additional support.

The cavern should be located to minimize the rock defects (faults and friable bedding planes) in its vicinity. Where such zones are unavoidable, the galleries or drifts may be located in such a manner that additional support can be placed, as required, in the vicinity of the weak zones.

TABLE 3-1. CHARACTERISTICS OF LARGE CAVERNS

Location, Dimensions and Depth of Cavern	Rock Properties	Support Used	Observed Displacements and Behavior
Waldeck II Powerhouse *Edersee, West Germany* Elliptical cross-section height = 177' width = 110' length = 348' Depth = 864' ($\sigma_h = 0.4\ \sigma_v$) Ref. 1, 2, 3.	Interbedded graywacke and clay shale *Average dip of bedding 32°* Faults and joints form wedges in roof Predominantly graywacke in roof: q_u = 11,400 psi E_{field} = 1.1 x 10^6 psi Predominantly shale in sidewalls: q_u = 7,100 psi E_{field} = 0.4 x 10^6 psi E_{field} from plate load tests, radial load test in gallery and borehole dilatometer.	Deep prestressed rock anchors for primary *support in crown and sidewalls*: 375 kip capacity - prestressed to 291 kips 4 1/2 in. dia. borehole length = 80' (20' grouted at end) *spacing* = 13.1' x 9.8' Short rock bolts for intermediate support: 26.5 kip capacity length = 20' (crown) 30' (sidewalls) spacing = 26 ft²/bolt Shotcrete 8-10 in. thick, reinforced with wire mesh.	Crown: 0.6-0.8 in. (max value of 1.5 in. occurred in a fault zone) Sidewalls: 0.4-0.5 in. (max 0.6 in) Most of the deformation occurred *within 20 ft of the cavern surface.*
Hongrin Powerhouse Veytaux, Switzerland Essentially semi-circular cross-section: height = 88' width = 100' length = 450' Depth = 214-492' Ref. 4, 5.	Marly limestone and limestone-schist Bedding sub-horizontal, undulating Several groups of sub-vertical fractures, badly fractured rock and crushed zones present q_u = 14,200 psi $E_{seismic}$ = 3.3-4.7 x 10^6 psi E_{field} = 0.7-1.4 x 10^6 psi (plate load tests) E_{field} = 1.4 x 10^6 psi (average of 205 measurements of bearing plate displacement during anchor load tests).	Deep prestressed rock anchors for primary support in crown and sidewalls: 375 kip capacity - prestressed to 310 kips (crown) 300 kip capacity - prestressed to 250 kips (sidewalls) 4 1/2 in. dia. borehole length = 43' (crown) 36' (sidewalls) (10' grouted at end - subsequently fully grouted) spacing - 14.1' x 9.5' (crown) - 450 ft²/anchor (sidewalls) Some additional 65' anchors used in crown in poor rock zones Short rock bolts for intermediate support in crown: tensioned to 35 kip working load and subsequently fully grouted length - 13' (2' epoxy resin at end) spacing - 60 ft²/anchor Shotcrete 6 in. thick reinforced with wire mesh.	Crown: 0.4-0.8 in. in poor rock less than 0.2 in. in good rock sections.
Portage Mountain Powerhouse *British Columbia, Canada* Rectangular cross-section with parabolic arch: height = 153' width = 67' length = 890' Depth = 450' ($\sigma_h = 2\sigma_v$) Ref. 6.	Interbedded sandstones, siltstones, shales and coals (dip of bedding 5-10°) *Jointing in siltstones and shales;* gouge seams in some coal and shale horizons due to slip along bedding planes Roof in shale member: q_u = 16,000 psi normal to bedding q_u = 13,000 psi parallel to bedding E_{lab} = 4.4-5.6 x 10^6 psi Sidewalls in sandstone member: q_u = 20,000 psi normal to bedding q_u = 18,000 psi parallel to bedding E_{lab} = 2.2-2.6 x 10^6 psi.	Mechanical rock bolts in crown, tensioned to 2/3 yield and subsequently full grouted: *23 kip yield strength* length = 14' 20' in center of arch spacing - 5' x 5' Reinforced concrete arch.	Crown: NW half of cavern 0.4 in. average (0.9 in. max) SE half of cavern 2.5 in. average (5.8 in. max) Large displacements due to opening up of horizontal bedding surfaces in a zone extending more than 20 ft above the crown.
Drakenberg Pumped Storage Scheme, South Africa Machine Hall: flat roof with 45° haunches: height = 100' width = 55' length = 650' Depth = 500' ($\sigma_h = 3\sigma_v$) Ref. 7	Flat-lying interbedded siltstones, mudstone, sandstone. Siltstone, mudstone near roof of machine hall: q_u = 3400 to 13,000 psi E_{lab} = 3 x 10^6 psi	Long rock bolts: *Dywidag bars encased in* corrosion protection system, tendioned to 66 kips, grouted later length = 20' to 23' spacing = 8' x 8' (Corrosion protection system: corrugated plastic tube with cement grout filling between bar and tube) Short rock bolts: resin anchorage, tensioned to 11 kips length = 10' spacing = 4' x 4' Shotcrete and wire mesh.	*Crown:* Small (1/16") offsets along horizontal bedding planes in haunch below crown. Tendency for slabbing along bedding in flat roof. Poor bond between shotcrete and *bedding plane partings in roof.*
Poatina Power Station Tasmania Flat roof, 45° haunches: height = 85' width = 45' length = 300' Depth = 500' (σ_v = 1200 psi σ_h = 1800-2400 psi) Ref. 8	Thin to massive bedded mudstone (thin bedded in roof, massive in walls) Bedding horizontal q_u = 5,000 psi Perpendicular to bedding: E_{lab} = 4.3-5.3 x 10^6 psi* E_{field} = 2.4 x 10^6 psi (flat-jack tests) Parallel to bedding: E_{lab} = 6.1-6.4 x 10^6 psi* E_{field} = 3.2 x 10^6 psi (flat-jack tests) * air-dried samples	Fully grouted rock bolts in crown and sidewalls: length = 14' (crown) 12' (haunches and top of walls) *spacing* = 3' x 3' Bolts doubled in crown, using two 3' x 3' patterns, to hold slabbing rock 14" bolts used in sidewalls at midheight to prestress rock in potential tensile zones 4" gunite and wire mesh in crown.	Crown: 0.10 in. (0.18 in. after shear developed) Sidewalls: 0.4 in. 3' deep stress relief slots used in crown to reduce compressive stresses and slabbing of beds in crown. Shear failure developed on horizontal bedding plane at intersection of haunch and crown, resulting in 1/8 in. displacement of haunch into cavern.

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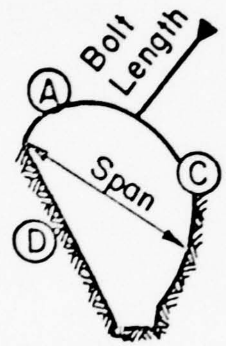
TABLE 3-1. CHARACTERISTICS OF LARGE CAVERNS (continued)

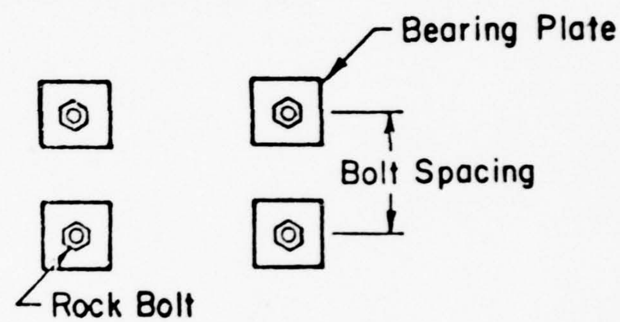
Location, Dimensions and Depth of Cavern	Rock Properties	Support Used	Observed Displacements and Behavior
Coo Pumped Storage Station, Coo, Belgium height = 141' width = 69' length = 420' Depth = 260-330' Ref. 9, 10	Interbedded quartzite and phyllades Bedding near vertical, average dip 70° Jointing sub-vertical Heavily faulted, some zones badly fissured and crushed $E_{field} = 0.2\text{-}0.4 \times 10^6$ psi (from borehole pressuremeter)	3 ft thick concrete placed in sections as crown was excavated Fully grouted, high strength rock bolts used to support sidewalls: 72 kip working load (2/3 of ultimate) length = 25-33' (some as long as 70') spacing = 270 ft^2/bolt (varies with rock quality from 115 to 290 ft^2/bolt)	Sidewalls: average about 5 in. near base of sidewalls
Dehradun Powerhouse India height = 105' width = 72' length = 361' ($\sigma_h = \sigma_v$ = 640 psi) Ref. 11	Slates interbedded with thinly laminated limestones Complex structure containing slip planes, shear seams, joints and faults Slip planes form wedges in sidewalls	Sidewalls supported by 132 kip anchors from galleries outside the cavern. The cavity was benched down in 10 ft stages, exposing the pre-drilled anchor holes. The anchors were then fitted with base plates, prestressed and subsequently fully grouted. Shotcrete 1.5-2.5 in. thick was used for immediate support prior to prestressing of the anchors.	Sidewalls: less than 0.25 in. subsequent to installation of anchors Surface slabbing in thin sheets resulting from distressing.
Nevada Test Site Cavities I and II Rainier Mesa, Nevada 120-ft-diameter hemispheres with plane surface inclined 72° from horizontal: height = 140' width = 80' length = 120' Depth = 1300' (σ_v = 1000 psi) σ_h = 500 psi) Ref. 12, 13.	Massive, bedded tuff RQD: 95-100% q_u = 1500 psi $E_{lab} = 0.5 \times 10^6$ psi $E_{lab\ sonic} = 1.5 \times 10^6$ psi $E_{seismic} = 1.0 \times 10^6$ psi	Rock bolts, 1 1/8 in. dia., tensioned to 50% of yield: length = 32' (crown) 24' (walls) (8' grouted at end - remainder ungrouted) spacing = 3' x 3' (crown) 6' x 6' (walls) Additional bolts required to stabilize Cavity II wall: length = 48' spacing = 3' x 3' Guinte added to prevent drying and cracking in crown, after excavation was completed.	Elastic, crown and sidewalls: 0.15 to 0.4 in. Displacements due to shallow slabbing in crown at depth of 3 to 5 ft 1 to 2 in. Cavity I wall stable under design bolting. Cavity II had joints and bedding planes intersecting wall which formed an unstable wedge. Deep-seated displacements of 1 to 2 in. occurred at 10 to 30' depth over 80' x 100' area of wall. Stabilized with additional bolts.

TABLE 3-2. ROCK BOLTS IN CAVITIES I AND II

	Equivalent Cavity Span	Tensioned Bolt Load	Bolt Spacing	Bolt Length	Bolt Length / Cavity Span	Bolt Pressure P_1*
A. Dome	100'	30,000 lb	3' x 3'	32'	0.32	20 psi
C. Curved Surface	100'	30,000 lb	3' x 3'	24'	0.24	20 psi
D. Plane Face						
(1) Cavity I and Cavity II, Prior to Failure	140'	30,000 lb	6' x 6'	24'	0.17	5 psi
(2) Cavity II, During Failure	140'	60,000 lb	6' x 6'	24'	0.17	10 psi
(3) Cavity II, After Stabilization	140'	30,000 lb	3' x 3'	48'	0.33	20 psi

* $P_1 = \frac{\text{Tensioned Bolt Load}}{144 \times (\text{Bolt Spacing})^2}$





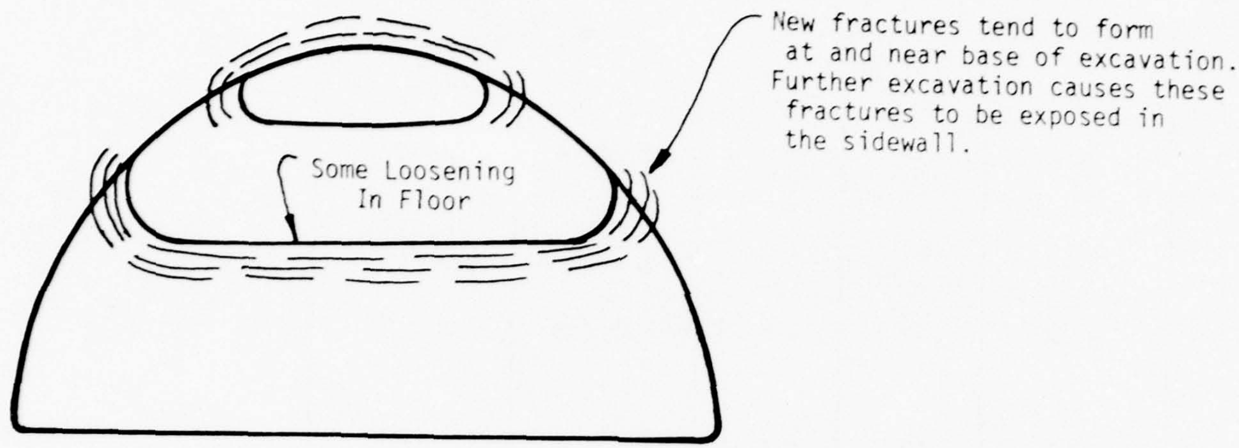
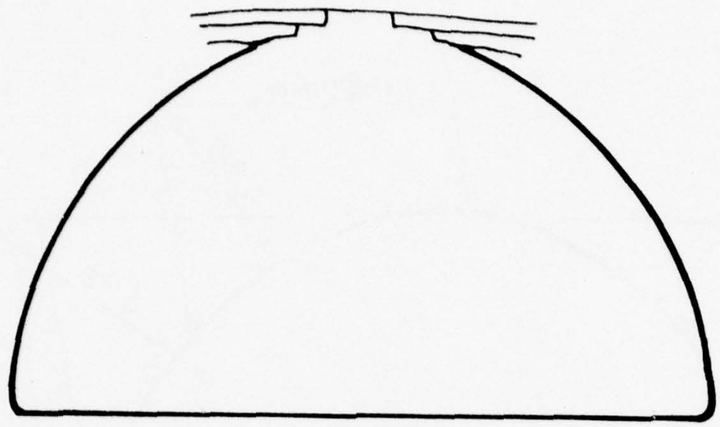
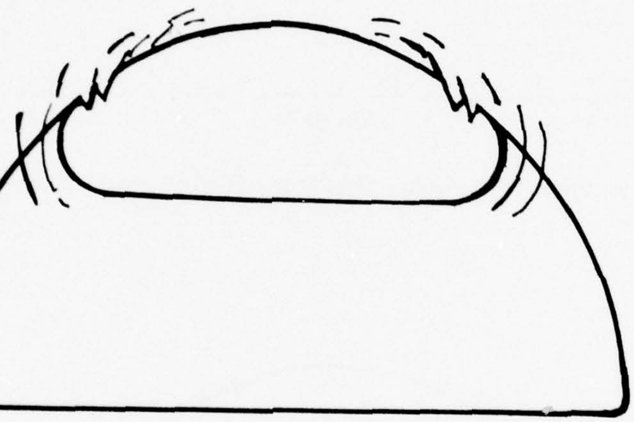


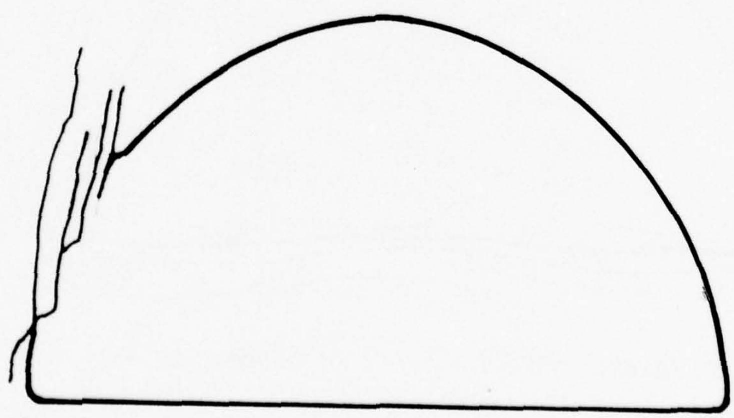
FIGURE 3-1. FORMATION OF NEW SURFACE FRACTURES DUE TO CAVITY EXCAVATION



(a) Loosening along bedding plane partings.

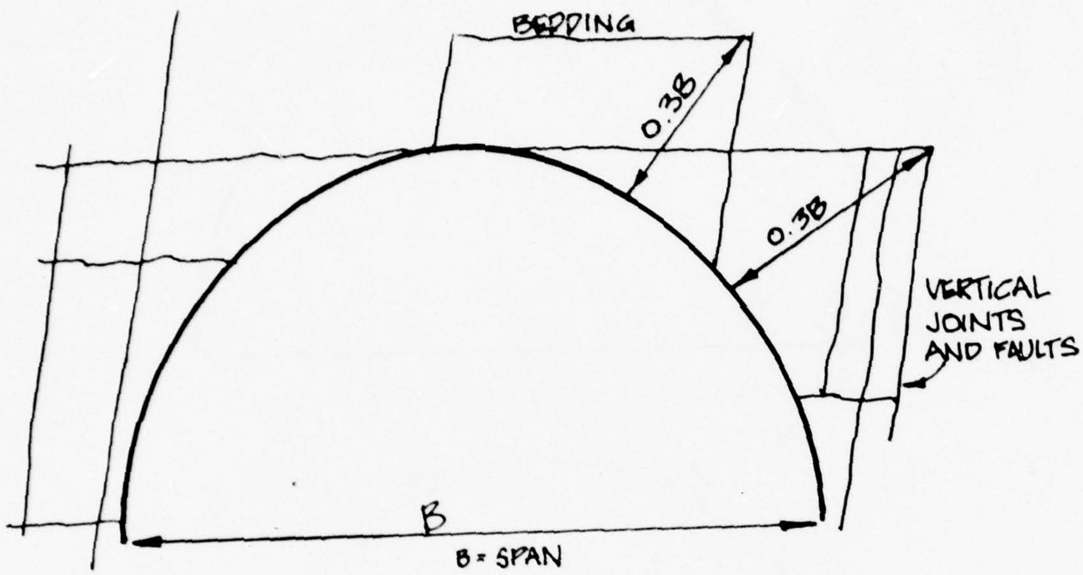


(b) Loosening, fallout, overbreak along new fractures.



(c) Local loosening, slabbing, and overbreak where walls are parallel and adjacent to vertical faults.

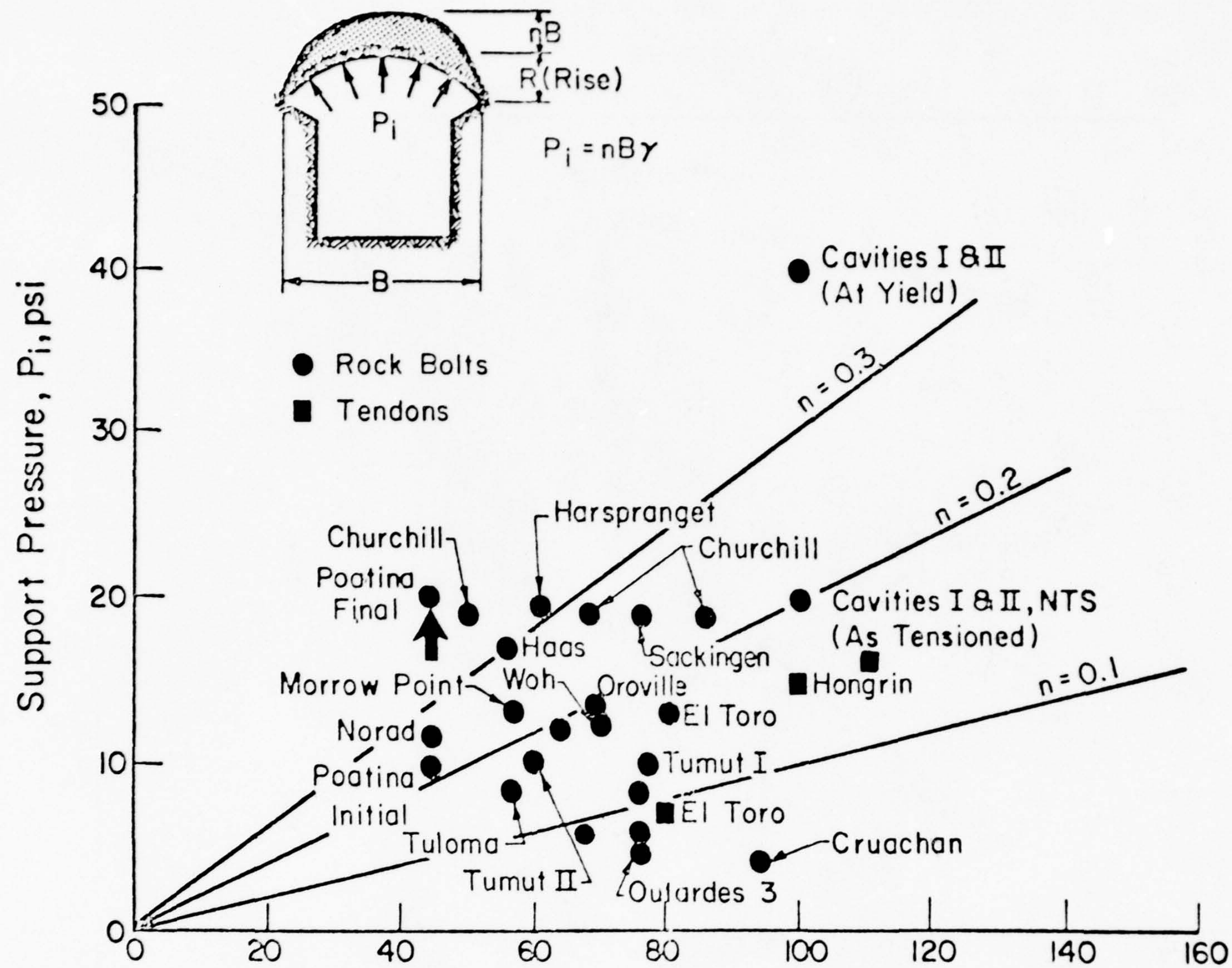
FIGURE 3-2. POTENTIAL ROCK FAILURE MODES DURING CAVITY EXCAVATION



(d) Large wedges formed by bedding, faults, and new fractures.

(e) Invert loosening, loss of support at wall.

FIGURE 3-2. POTENTIAL ROCK FAILURE MODES DURING CAVITY EXCAVATION
- continued -



WIDTH OF OPENING, B, FEET

FIGURE 3-3. SUPPORT PRESSURES USED IN CROWN OF CAVERNS

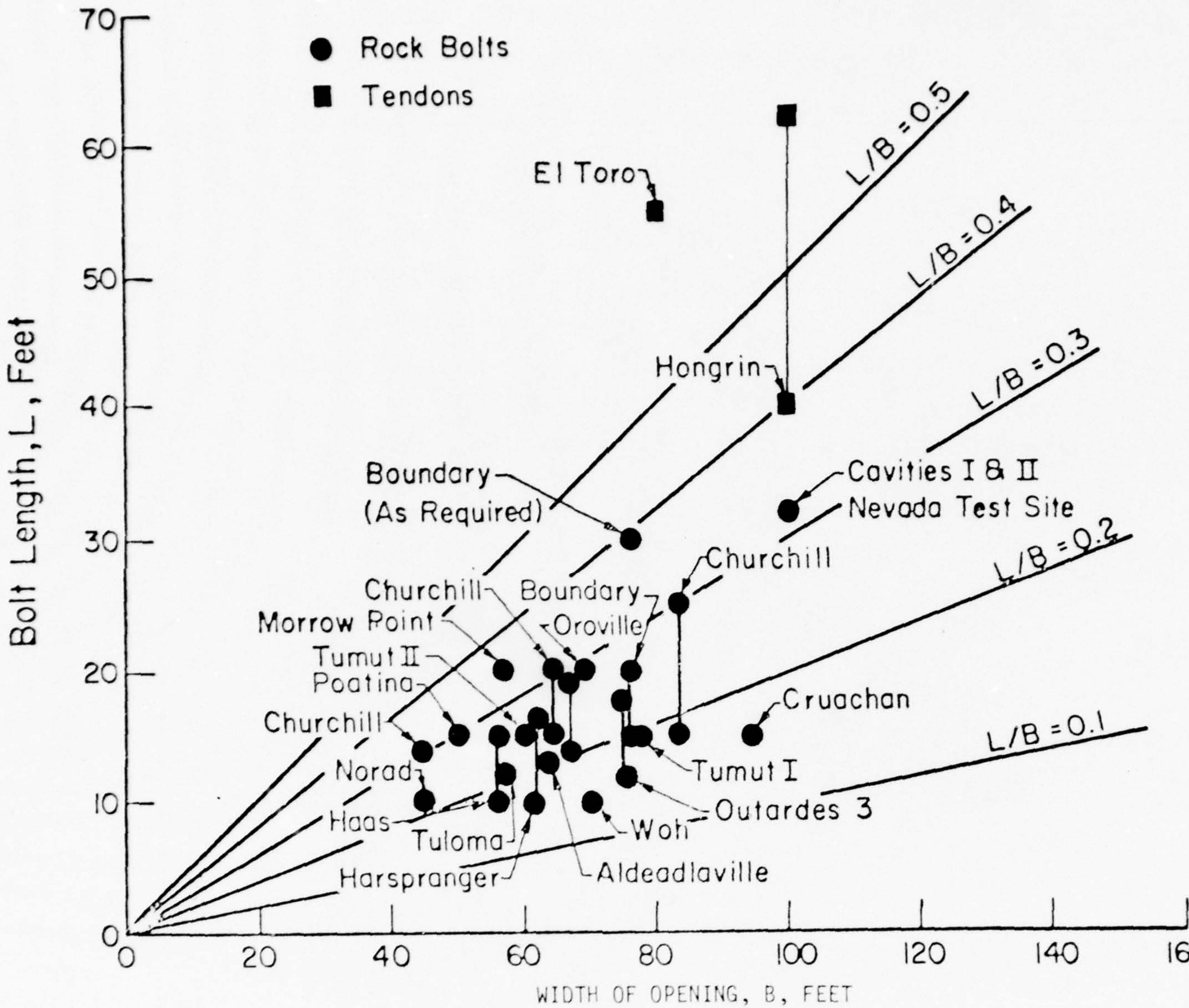


FIGURE 3-4. BOLT LENGTHS USED IN CROWN OF CAVERNS

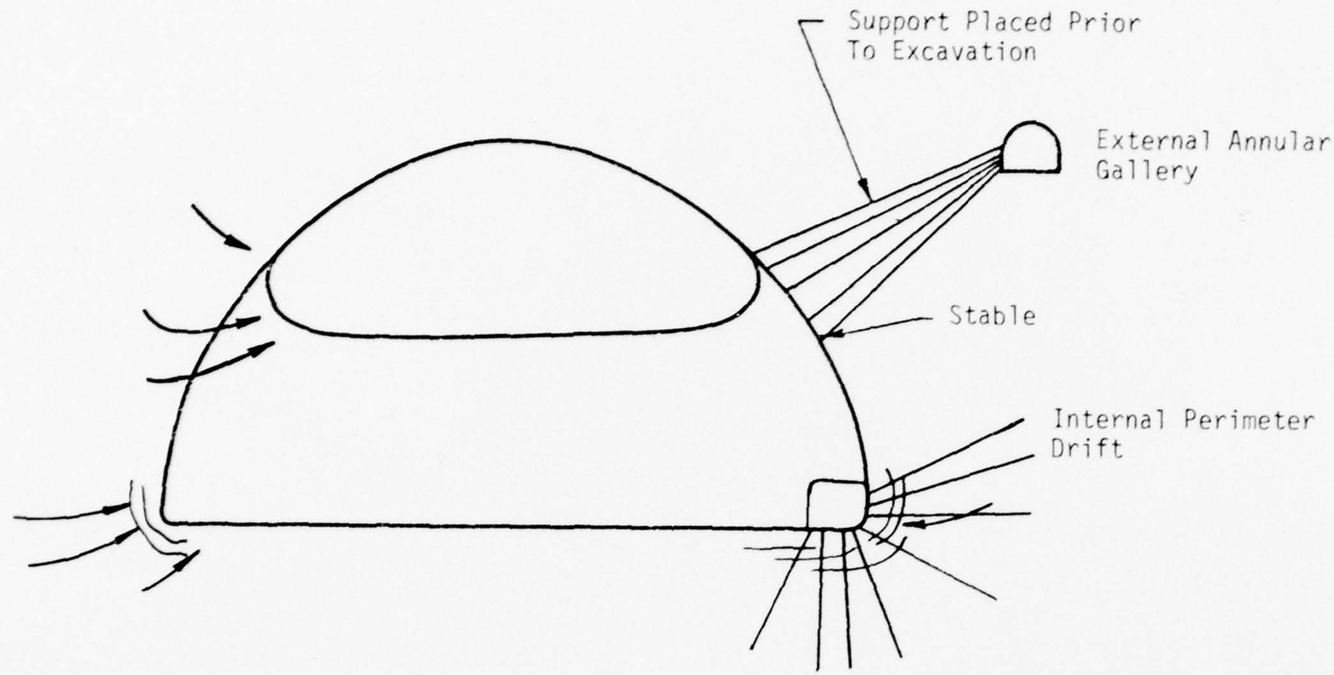


FIGURE 3-5. PRE-SUPPORTING OF ROCK MASS TO LIMIT INWARD MOVEMENT AND FAILURE AT BASE OF EXCAVATION

4. GENERAL DESIGN CONSIDERATIONS

This section describes the important engineering aspects considered in the preliminary design of the large hemispherical cavity for Rainier Mesa. The recommended orientation and final shape of the cavity is discussed together with the effects of intersections. The use of annular tendon galleries for cavities larger than 45.7 (150 ft) in diameter is described and the advantages and disadvantages in their use is also discussed. The general approach to the design of the rock support systems for both internally supported and externally supported (with annular tendon galleries) cavities is presented including a discussion pertaining to the monitoring of tendon loads and rock stability.

4.1 SHAPE AND ORIENTATION

In order to meet the functional requirements for their use at NTS, it has been determined that the caverns must include a large planar surface and must be designed such that the minimum radius from the center of the planar surface to any point on the cavity wall is maximized. A hemispherical shaped cavern achieves both these goals while requiring a minimum excavated volume and therefore represents the starting point for shape considerations. It is recommended that the upper portion of the chamber be modified from the true hemispherical configuration by forming a matching paraboloid as shown in Figure 4-1. This will result in a smaller arch radius at the top of the cavity and is desirable since the radius is reduced in an area where flat slabs would otherwise tend to develop along bedding plane surfaces. It also provides a more equidimensional cavity shape during initial excavation stages so that a top down-benching excavation procedure would not be initiated under a flat roof.

Although some early rock mechanics references placed emphasis on the significance of the maximum tangential stresses at the boundary of openings,

it should be recognized that these stresses are not the most critical factors in determining the shape and stability of an opening. Of more significance are the depth of the rock zone influenced by increased stresses due to the presence of the opening and the location of joints and bedding plane weaknesses which govern the geometry of wedges that can potentially displace into the cavern. For both of these considerations it is desirable to have small radius surfaces rather than large planar faces. In this way the depth of the rock zone which is highly stressed or which is bounded by joints that could form unstable wedges is minimized. For example; high, tangential stresses will develop around tight radius corners in a cavern, but their effect on stability is minor because the depth of the highly stressed zone is small. On the other hand, large radius surfaces, such as planar walls, will have lower tangential stresses at the surface, but the depth of the zone having low confining pressures will be large, so that relatively high shear stresses will extend to a large depth. In general, large planar surfaces should be avoided, particularly when they are almost parallel to major joint or bedding discontinuities or are parallel to the direction of the maximum free-field stress.

There are several reasons which make it advantageous to locate the plane surface horizontally at the bottom of a hemispherical cavern:

1. No planar surface is present in the upper portions of the cavern from which rock blocks can loosen under the influence of gravity.
2. As the cavern is excavated downward a relatively planar bottom is desirable for ease of working. Thus, the planar bottom of the hemisphere fits the shape that would normally be used at intermediate excavation stages. Ease of surveying, increased working area, and increased volume of excavation per unit area of wall are all advantages of the flat bottom configuration.
3. From the standpoint of the stress distribution around the opening, there is no advantage to placing the planar surface of the hemisphere at an angle to the horizontal. Since the natural vertical stress in Rainier Mesa is high, and one of horizontal stresses is low, orienting the plane surface horizontally (parallel to the minimum stress) would be one of the most desirable configurations.

Referring to past experiences, the plane faces of the Red Hot/Deep Well caverns were oriented steeply so that the end of the LOS pipe was accessible from a drift at tunnel level. However, there was no advantage from the standpoint of stability or constructability for such a configuration; in fact construction was more difficult and the stability in Deep Well (Cavity II) was adversely affected by the presence of joints oriented almost parallel to the plane face. Therefore, from the standpoint of the constructability and stability of the chamber, it is recommended that the modified hemispherical shape be used, with the plane surface located at the bottom.

It is anticipated that during the excavation of the cavity, access or cross-cut drifts on the order of 3.35 m (11 ft) high by 3.96 m (13 ft) wide will intersect the surface of the hemispherical cavity. However, penetrations of this size have no effect on the shape considerations of the cavern and will not produce any significant support problems. Normal portal rockbolting should be installed around the perimeter of the drift to prevent the loosening of wedges and slabs around the intersection.

4.2 ANNULAR TENDON GALLERIES AND PERIMETER DRIFTS

A review of the documentation of previously excavated caverns indicates that the vast majority of these employed an internally installed rock support system. Such a method usually involves either the mining of perimeter drifts or the excavation of the cavity surface by a top down-benching procedure followed by the placing of rockbolts or tendons as the wall is exposed. This procedure is quite adequate for cavities smaller than approximately 45.7 m (150 ft) in diameter. In such cases the depth of rock located above the cavity surface which is influenced by increased stresses due to the presence of the opening is small and rock fracturing and rock movement prior to the installation of the rockbolts or tendons is not significant. However, as the cavity radius increases, the depth of influence also increases and the problem of rock fracturing and movement becomes significant particularly near the crown of the cavern and immediately below the cavern floor as it is excavated down-

ward. Therefore, as discussed in Section 3.3, external tendon/instrumentation galleries are basic to the design of the larger caverns. These annular tunnels or "doughnut drifts" are mined outside the perimeter of the hemisphere as shown in Figure 4-2 and serve as an access area for the installation of tendons to support the cavern wall prior to general excavation, for observing rock movement and monitoring tendon loads from a location which is accessible throughout the period of excavation, and for installing additional support at any time during excavation if the measurements indicate excessive rock movement. As a result, the external annular drift scheme provides more control than is possible with a more conventional excavation and support scheme. The use of the annular tendon galleries increases the cost of initial development and the time frame prior to beginning cavity excavation. However, some of the cost and time is regained due to more efficient cavity mining once it is begun. At least one external annular gallery is recommended for caverns 46 m (150 ft) or more in diameter. In small caverns, the use of external galleries may not be feasible since the cost and time to mine the annular gallery approaches that of the entire cavity excavation.

Internal perimeter drifts are also recommended at the permanent floor of the cavern, at higher elevations where external perimeter galleries are not used, and where critical rock support conditions exist. The internal perimeter galleries are feasible in caverns smaller than 46 m (150 ft) in diameter, as well as in larger caverns. Support installed in the wall or floor from the perimeter drift will limit rock movements. Instrumentation can also be placed from the perimeter drifts so that rock movements can be observed throughout the entire general excavation period. However, access to the instrumentation will be more difficult than in the external annular galleries, and the instruments must have good protection from damage during construction. Both external galleries and internal perimeter drifts can be utilized in a single chamber, as shown in Figure 4.2.

4.3 ROCK SUPPORT SYSTEMS

The overall rock support for a large underground cavity includes primary and secondary support systems and rock movement and support load monitoring systems. The primary system consists of long rockbolts or tendons while

the secondary system consists of short rockbolts, wire mesh and shotcrete. The combined rock support system for a large chamber is designed to:

1. Support the large wedges of rock that are formed by intersecting vertical joints, bedding plane weaknesses and new fractures within the region of influence.
2. Minimize the formation of new fractures and the loosening of slabs bounded by fractures formed as a result of high stress concentration near the cavity surface.
3. Support the shallow fractured rock zone.
4. Prevent deterioration, drying, and cracking of the rock surface that would result in fallout of blocks or loss of bearing of rockbolts.
5. Prevent excessive cracking and heave in the temporary or final inverts during the various stages of excavation which could lead either to a stability problem in the side walls or to loosening of rock below the permanent floor that would make it unsuitable for the user's experiments.

It should be noted that the depth of the loosened zone on the cavern walls will be influenced by the excavation method. Mechanical excavators will reduce rock loosening with respect to that obtained with blasting. If blasting is used at the perimeter of the excavation, the smooth wall blasting technique should be employed where lightly loaded, closely-spaced perimeter holes are drilled parallel to and on line with the wall of the cavern. This method will result in less damage than would be obtained with non-smooth blasting procedures. Overbreak and loosening will also be minimized if the surface is cut smoothly without large reentrants or overbreak.

Excavation control alone, however, will not minimize loosening. Placement of support prior to or immediately after excavation is reqired in order to minimize loosening. The depth of the loosened zone will be greater the larger the unsupported span and the larger the size of the chamber. In the Red Hot/Deep Well caverns, major loosening occurred within .91 m (3 ft) of

the wall and minor loosening up to 2.44 m (8 ft) behind the wall. In the proposed 73.2 to 91.4 m (240 to 300 ft) diameter caverns, it is anticipated that the major loosened zone can be kept within 1.52 m (5 ft) of the wall, with some new fracturing and minor loosening up to 3.05 m (10 ft). Appendix B presents a solution for determining the depth of the plastic zones behind the wall of a spherical chamber in a uniform stress field. The solution can be used to obtain an approximate estimate of the zone of fracturing behind the curved walls of a hemispherical chamber in tuff.

4.3.1 Primary Support Characteristics

The primary support for the cavern is provided by high capacity tendons or rockbolts that tie the rock mass back and permit the rock to support itself. The design approach is to provide rockbolts or tendons of sufficient length and capacity to hold the weight of the large, potentially unstable rock wedges in place. It is assumed that the support is installed in such a way that loosening of large wedges of rock is minimized and therefore, support pressures are also minimized. Table 4-1 summarizes the design rock pressures to be supported, and the lengths for the rockbolts and tendons. Note that the support pressures for the 36.6 m (120 ft) diameter cavern are consistent with those used on the 36.6 m (120 ft) diameter Red Hot/Deep Well cavern surfaces. The support pressures in the larger caverns increase in proportion to cavern width, as would be expected when supporting rock wedges of similar geometry that are loaded by their own weight.

In the upper portion of the cavern, the high capacity rockbolts and tendons will have lengths of between 30 and 40% of the cavern diameter. Because of the large length of hole required for the tendons and rockbolts, it may slow excavation to install them from within the chamber. The annular galleries provide the access from which the tendons holes can be drilled and the tendons installed. Tendons can be in place prior to excavation of the cavern, thus providing cavity wall support at and below the bottom of the excavation even before it is exposed. In this way loosening of the side and bottom will

be minimized. It is felt that the stability problem on the plane face of Deep Well cavern (Cavity II) may have been aggravated by loosening below the bottom of the excavation, prior to installing support. Tendons are preferable to high capacity coupled reinforcing bars in the annular gallery because they are easier to install from a constricted area.

Bearing pads in the annular gallery will carry the tendon loads. If it is determined that elastic rock movements during cavern excavation will be large enough to overstress the tendons, a compressible bearing surface can be used. Such a bearing surface would be designed to deform sufficiently with increased tendon load to minimize the build up in tendon stress so that loads remain relatively constant with rock displacement. The bearing pad for the tendons could also be designed to serve as a load cell, by calibrating the deflection of the pad to the force in the tendon.

The end of the tendon that is adjacent to the cavern wall will be anchored with a cement grout for a length of 4.6 to 9.1 m (15 to 30 ft) and should extend to within approximately .3 to .9 m (1 to 3 ft) of the wall of the cavern. The tendons will be tensioned but will be non-grouted or grouted with a bond breaker. Each tendon will be preloaded to a value approximately 20 to 30% above its lock-off load. Lock-off loads, below the design tendon load may be specified if it is determined that elastic rock movement during the excavation will stress the tendons. The tendon lock-off load divided by the square of the spacing of the tendons on the cavern surface is equal to the design pressures given in Table 4-1. Emergency adjustment in tendon load can be made from the galleries during the cavern construction. If it is determined that adjustment in tendon loads or measurement of tendon loads is not necessary, then it may be feasible to fully grout the tendons.

In the areas where external tendons are not used, either tensioned tendons or reinforcing bars may be installed from within the cavern. Fully grouted rock bolts or tendons are desirable, since they do not require maintenance of the bearing surface in the cavern to remain effective. Bearing pads

in the cavern must be supplied with a properly mortared bed and must be sufficiently large in order to prevent cracking and failure of the rock bearing surface.

4.3.2 Secondary Support Characteristics

In addition to the externally installed high capacity tendons and rockbolts, short rockbolts on the order of 3 to 4.6 m (10 to 15 ft) long are required to support the rock immediately adjacent to the wall of the cavern. Short rockbolts are necessary since the ends of the tendons installed from the external annular galleries may vary in location by several feet and will end up to a distance of .9 m (3 ft) behind the wall. Also, long bolts installed internally from perimeter galleries at the permanent wall will be clustered so that the space between the bolt clusters will require support by short bolts.

The surface bolts are to be non-tensioned, fully grouted with resin cartridges and installed on a 1.5 by 1.5 m (5 by 5 ft) pattern immediately upon excavation of the permanent wall surface. Welded wire fabric with a 10 by 10 cm (4 by 4 in) mesh is to be attached to the surface bolts as excavation is carried downward. A thin shotcrete layer, approximately 5 to 8 cm (2 to 3 in) thick, will then be applied to minimize cracking and drying of the tuff.

4.3.3 Stability Monitoring

Figure 4-3 illustrates some of the monitoring systems which can be employed in the cavern. The primary method for monitoring stability is by means of borehole extensometers. It is recommended that the extensometers be installed from the external annular galleries. In this way the extensometers would be easily accessible throughout the cavern excavation, additional extensometers could be installed if required, and the extensometers would be in place prior to cavern excavation so that the entire displacement history could be recorded. Extensometers can be installed and read from the cavity interior and are valuable because they can provide warning of impending difficulties before they can be recognized by the naked eye. Procedures for evaluating

stability by means of extensometers is outlined in Appendix C. Large movements or rates of movement may be indicative of developing instability. Extensometer displacements may approximate the movements expected from elastic theory, if loosening of the rock is minimized. Figure 4 in Appendix C illustrates the movements observed in the Red Hot/Deep Well caverns. Total elastic displacements of approximately 5 cm (2 in) should be expected in a 91.4 m (300 ft) diameter opening between the external tendon gallery and the cavern wall.

The performance of the tensioned tieback system should be monitored with load cells, such as the bearing pad devices described briefly in Section 4.3.1. These units can be used to determine whether or not anchorages are maintaining the tendon load, or alternatively, if excessive loads are developing in the support system.

Other possible methods of displacement monitoring include:

1. Periodic sonic logging and borehole viewing by means of borescopes in holes drilled internally or from the annular galleries. The development of new fractures, open joints, and lateral offsets can be observed in boreholes installed prior to general excavation. Periodic logging of the holes visually or with other geophysical tools, will provide an indication of the progression of fracturing and loosening throughout the excavation period. As the minimum, careful visual observations of borehole offsets and opening of fractures should be made in boreholes that extend through the zone immediately around the cavern. Such examinations have been successfully used in the Red Hot/Deep Well caverns and in the Drakensberg Pumped Storage Chamber, where stresses were high.
2. Seismic refraction profiling along the cavern wall to determine the depth of the loosened zone. This work would not

provide the daily control that is provided by the borehole extensometers, but would serve to confirm the extent of loosening that has developed at intermediate stages in the cavern construction.

3. Stress meters embedded in boreholes, installed prior to excavation. Changes in stress can be observed as the excavation approaches and passes the locations where the stress meters are installed. The stability of a cavern is more difficult to interpret with a stressmeter than with a borehole extensometer, but the instrument can provide useful information when used in conjunction with borehole extensometers.
4. Rock noise monitoring from the perimeter drifts to locate zones of active rock fracturing.

TABLE 4-1. RECOMMENDED DESIGN PRESSURES AND TENDON/ROCK BOLT LENGTHS

Cavern Diameter ft	Type of Support	Rockbolt/Tendon Capacity Kips	Support Pressure Arch psi	Bolt or Tendon Length	
				Arch ft	Sidewall ft
80	Internal	53.3	13	28	16
120	Internal	80	19	40	24
RH/DW 120	Internal	80	23	36	24
160	Internal	107	26	55	32
180	Internal	120	29	60	36
180	External	120	29	75-85	36
240	External	160	39	95-105	48
300	External	200	48	120-130	60

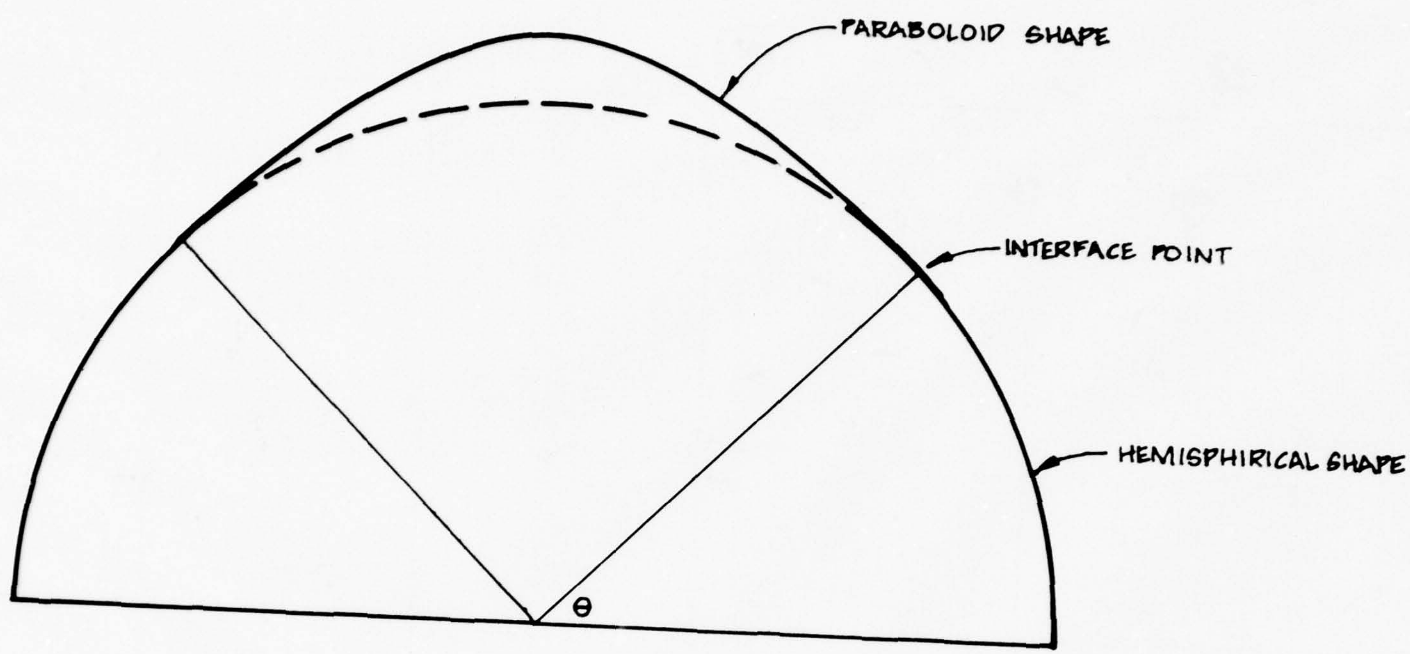


FIGURE 4-1. GENERALIZED LARGE CAVITY SHAPE

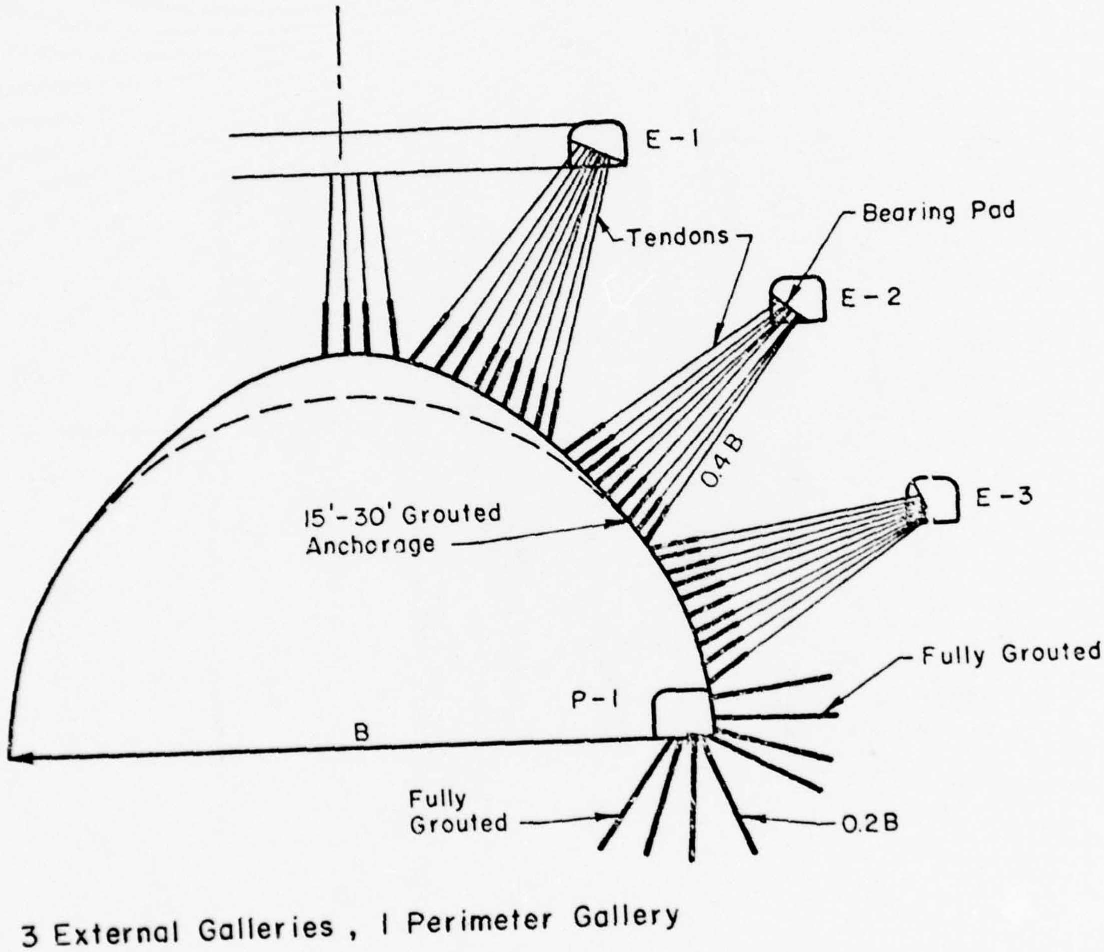


FIGURE 4-2. LARGE CAVITY CONFIGURATION WITH ANNULAR TENDON GALLERIES AND A PERIMETER GALLERY

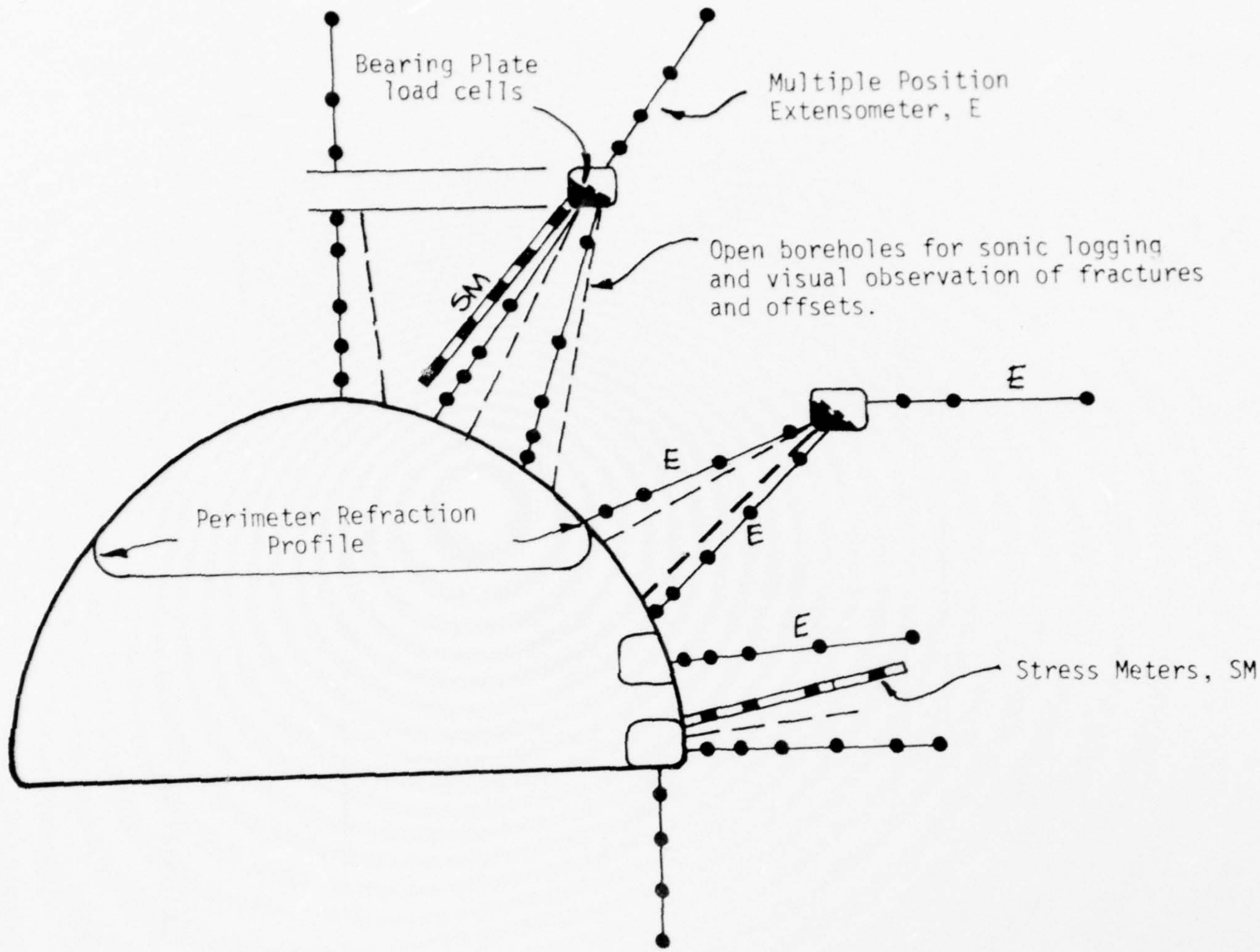


FIGURE 4-3. ROCK DISPLACEMENT MONITORING SYSTEMS

5. CAVITY DESIGN DETAILS

This section contains the basic cavity design details and the preliminary rock support system design for each of the cavity sizes for which cost estimates have been prepared. These details form the bases for estimating the cost and schedule (working days) required to construct the cavities. The general dimensional data for the cavities is presented followed by a detailed description of the rock support requirements for each.

In order to clearly depict the cost and schedule variations between internal and external rock support designs, this report only evaluates cavities which are either totally internally supported or totally externally supported. However, as discussed in Section 4, it is recommended that caverns larger than 45.7 m (150 ft) in diameter be designed with at least one external gallery from which to support the crown of the chamber prior to general excavation. In addition, internal perimeter galleries from which rock bolt support is placed prior to general excavation are also recommended. Therefore, an alternate rock support approach is presented which combines the use of externally installed tendons and internally installed rockbolts. The advantages of this alternate approach are also discussed.

5.1 DIMENSIONAL DETAILS

As discussed in Section 4, the cavity shape is based upon superimposing a paraboloid of revolution and a hemisphere. An angle of 45° was selected for the matching line between the two arcs and the resulting cavity generalized dimensions are shown in Figure 5-1. Based upon the definition of terms shown in Figure 5-1 the dimensional data for each cavity is presented in Table 5-1 and includes arc length (S), surface area (A) and cavity volume (V) computed by the following formulii:

Spherical Zone

$$S_1 = \pi D/4 = 1.570796R$$

$$A_1 = \pi D H_1 = 4.442883R^2$$

$$V_1 = \frac{\pi H_1}{24}(3D^2 + 3L^2 + 4H_1^2) = 1.851201R^3$$

Paraboloid Zone

$$S_2 = L(1 + \frac{8}{3} R^2 - \frac{32}{5} R^4 \ldots) = 1.682093R$$

$$A_2 = \frac{2\pi L}{3H_2^2} [(\frac{L^2}{16} + H_2^2)^{3/2} - (\frac{L}{4})^3] = 2.048299R^2$$

$$V_2 = \frac{\pi H_2 L^2}{8} = 0.334758R^3$$

Total Cavity

$$S = S_1 + S_2 = 3.252890R$$

$$A = A_1 + A_2 = 6.491182R^2$$

$$V = V_1 + V_2 = 2.185959R^3$$

From these equations it can be seen that approximately 68.4% of the curvilinear surface area and 84.7% of the cavity volume lies in the spherical zone.

Also shown in Table 5-1 is the depth of cavity influence (equal to 0.2 x span) and a comparison of the surface areas and volumes of the various caverns using the 91.4 m (300 ft) span as the unit reference. It can be seen that the volumetric ratio for the 36.6 m (120 ft) cavity, which would correspond to the Red Hot and Deep Well chambers, is only 6.4% of that which would be contemplated for the 91.4 m (300 ft) cavity. Therefore, when assessing whether a cavity size is an extension of current practice or of the state-of-the-art, the area and volume ratios must be considered and not just simply the span ratio.

5.2 ROCK SUPPORT REQUIREMENTS

Before a discussion of the individual cavity designs, a few general comments concerning the rock support systems should be made. The primary support systems evaluated in this report consist of internally installed rockbolts or externally installed tendons to support the rock mass above the crown and surrounding the sides of the caverns. In addition rockbolts are installed in the floor and lower walls of the cavity to inhibit heaving of the floor and to prevent progressive spalling and loosening in the lower walls. The secondary support system consists of personnel rockbolts, wire mesh, and a layer of shotcrete which are installed to reduce the formation of new fractures, inhibit the loss of moisture in the exposed rock at the cavity surface, and protect against small rock falls.

The depth of rock surrounding the cavern which the rockbolts are designed to support is considered to be equal to 20% of the span. This rock load depth has a slightly parabolic shape such that the depth at the apex is equal to 20% of the span and thins as it progresses towards the base of the cavity. Therefore the internally installed primary rockbolts are specified to have lengths equal to 1/3 the span in the region between 0° and 30° from the vertical and lengths equal to 0.8 times 1/3 the span and 0.6 times 1/3 the span in the regions between 30° and 70° and between 70° and 90° from the vertical, respectively. Dywidag rockbolts are the assumed internal support system and catalog information concerning these rock anchors is included as Appendix D.

In order to assure that the anchor points for the external support configuration are located in rock which is unaffected by the presence of the cavity, the annular galleries are positioned at a minimum distance equal to twice the rock load depth (0.4 times the span) from the surface of the cavity. In addition, tendons installed from the annular galleries are situated such that they intersect the perimeter of the cavity at an angle not less than 60 degrees. The tendons consist of approximately 5 to 7 cables grouted together and catalog information concerning the tendons fabricated by the V.S.L. Corporation are included as Appendix E.

It is assumed for estimating purposes that the primary rock supports, whether in the form of rockbolts or tendons, are spaced on a 1.63 x 1.63 m (5.4 x 5.4 ft) pattern at the cavern wall. This value is based upon using 889,600 Newton (200 kip) capacity tendons for the 91.4 m (300 ft) chamber. For smaller cavity sizes, tendon spacing is held constant and capacity is reduced in proportion to the cavity span. Specific rock support sizes and capacities are given in the discussion of the individual cavity designs. For actual design of the smaller cavities, it may be desirable to increase spacing of the tendons and maintain a capacity of approximately 200 kips.

Since the externally installed tendons will only extend to within approximately 0.61 m (2 ft) of the cavity perimeter and will not support the surface rock, personnel safety rockbolts will be needed in addition to the primary supports. These rockbolts, which are a minimum of 2.22 cm (7/8 in) in diameter and 3.66 m (12 ft) in length, should have a minimum capacity of 111,200 Newtons (25 kips) and be installed on an approximate 1.52 x 1.52 m (5 x 5 ft) pattern. The personnel rockbolts are also to be used to support the 10.16 x 10.16 cm (4 x 4 in) wire mesh which covers the surface of the cavities.

In order to protect the exposed rock at the perimeter of the cavity from excessive loss of moisture and a resulting loss of strength, a 7.62 cm (3 in) layer of shotcrete is to be applied to the surface of the cavity after the wire mesh is in place. It should be a dry mix shotcrete with 1/2 coarse aggregate and with a minimum strength of 27,580 kPa (4000 psi) after a 28-day cure period.

In the subsections which follow, the details of the rock support system for each cavity size is discussed and illustrated. The size, capacity, and layout of the support system are described for each cavity including the drill lengths required for their installation.

5.2.1 24.4 Meter (80 ft) Internal Support Cavity

This internally supported cavity is the smallest considered in this report since it is well within current practice. The cavity layout is shown in Figure 5-2. The rock load depth for this cavity is 4.9 m (16 ft) and the rockbolt lengths are 8.5, 6.7, and 4.9 m (28, 22, and 16 ft) respectively in the three regions around the cavity. The floor is pinned by 4.9 m (16 ft) rockbolts. The design rockbolt capacity is 237,100 Newtons (53.3 kips) and 2.86 cm (1-1/8 in) re-bar rockbolts with a yield capacity of 266,900 Newtons (60 kips) are specified. A total of 376 rockbolts are required in the crown and sidewall of the cavity and 157 are required in the floor including additional rockbolt support around the corner. The rockbolts require a bore hole 4.45 cm (1-3/4 in) in diameter and a total drill length of 3144 m (10,316 ft) is necessary to accommodate them.

5.2.2 36.6 Meter (120 ft) Internal Support Cavity

This cavity design is the same size as the Red Hot and Deep Well caverns previously excavated in Rainier Mesa and represent the extent of current practice at the NTS. Figure 5-3 shows the layout of the cavity support systems which must support a rock load depth of 7.3 m (24 ft). The crown and sidewalls of the cavity are supported by rockbolts which are 12.2, 9.8, and 7.3 m (40, 32, and 24 ft) in length while 7.3 m (24 ft) rockbolts are used to pin the floor. Based upon the design rockbolt capacity of 355,900 Newtons (80 kips), high strength 2.54 cm (1 in) Dywidag "thredbar" rock anchors were selected which have a lock-off load capacity of 397,200 Newtons (89.3 kips). A total of 826 of these rockbolts are required to support the crown and sidewall while 270 are required to pin the cavity floor and corner. The borehole for the rockbolts is 6.03 cm (2-3/8 in) and a total drill length of 9545 m (31,316 ft) is required to accommodate the rockbolts.

5.2.3 48.8 Meter (160 ft) Internal Support Cavity

It was mentioned in Section 3 of this report that the extent of current practice in the excavation of underground machine halls would correspond to the excavation of a hemispherical cavern between 45.7 and 54.9 m (150 to 180 ft) in diameter. Therefore this cavity represents a design which is mid-

way in the range of what is considered current practice. This design also represents an excavated volume 2.3 times that for the Red Hot and Deep Well cavities. The layout of the support system is shown in Figure 5-4 which must support a rock load depth of 9.8 m (32 ft). The crown and sidewalls are supported by internally installed rockbolts which range in length from 9.8 to 17.1 m (32 to 56 ft) while 6.1 and 9.8 m (20 and 32 ft) rockbolts are employed to pin the floor. The design rockbolt capacity based upon the uniform 1.63 m (5.4 ft) spacing used for all cavities is 476,000 Newtons (107,000 kips) and 3.2 cm (1-1/4 in) Dywidag rock anchors with a lock-off strength of 584,100 Newtons (131.3 kips) were selected. A total of 1460 of these rockbolts are required to support the cavity crown and sidewalls while 483 are necessary to pin the floor and corner. A 6.5 cm (2-9/16 in) borehole is required for the rockbolts and a total drill length of 22,580 m (74, 080 ft) is necessary to accommodate them. In addition, a perimeter drift is included for the pre-supporting of the corner by means of rockbolts. The pre-supporting of the corner becomes of more importance as the cavity size increases and is considered necessary for caverns 45.7 m (150 ft) in diameter and larger.

5.2.4 54.9 Meter (180 ft) Internal Support Cavity

This cavity represents the largest hemispherical cavity which could be regarded as being within current excavation practice and is the largest internally supported cavity evaluated. The cost analysis for the construction of this cavity provides a cavern size overlap between internally and externally supported configurations but is somewhat academic since for rock stability reasons it is recommended that at least the crown be externally pre-supported prior to excavation. It is of interest to note that this 54.9 m (180 ft) cavity represents an excavated volume 3.8 times that of the cavities previously mined in Rainier Mesa. Figure 5-5 shows the layout of the cavity rock support systems which must support a rock load depth of 11.0 m (36 ft). The crown and sidewalls of the cavity are supported by rockbolts which vary in length from 11.0 to 18.3 m (36 to 60 ft) while 7.3 and 12.2 m (24 and 40 ft) rockbolts are used to pin the floor and corner. Based upon the design rockbolt capacity of 533,800 Newtons (120 kips), 3.2 cm (1-1/4 in) Dywidag rock

anchors with a lock-off strength of 584,100 Newtons (131.3 kips) were selected. A total of 1834 rockbolts are required to support the crown and sidewalls while 458 are required to pin the cavity floor. A 30,365 m (99,624 ft) total drill length of 6.5 cm (2-9/16 in) bore holes is necessary for rockbolt installation.

5.2.5 54.9 Meter (180 ft) External Support Cavity

It was discovered during the development of the external support concepts that completely external support of a cavity should be limited to caverns larger than approximately 48.8 m (160 ft) in diameter. This becomes apparent when comparing the excavated volumes for the annular drifts and for the cavity itself. For the smaller cavities the drift volume approaches that of the cavity and it becomes clear that the development costs become prohibitive for such a design. Therefore, cost and construction feasibility of externally supported cavities was only considered for those 54.9 m (180 ft) in diameter and larger. The support system layout for this cavern is shown in Figure 5-6 and includes two annular tendon galleries from which the tendons are installed and prestressed prior to general excavation of the chamber.

The rock load depth is 11.0 m (36 ft) and the tendon galleries are located at least twice that distance from the cavity surface. The crown and sidewalls are supported by tendons which range in length from 21.9 to 25.3 m (72 to 83 ft) while 7.3 and 12.2 m (24 and 40 ft) internally installed rockbolts are used to pin the floor and corner. The design support capacity is again 533,800 Newtons (120 kips), and tendon and Dywidag rock anchors with respective capacities of 551,100 and 584,100 Newtons (123.9 and 131.3 kips) are specified. A total of 1520 tendons and 314 rockbolts are required to support the crown and sidewalls and 458 are necessary to pin the cavity floor and corner. Since the tendons only extend to within approximately .6 m (2 ft) of the cavity surface it is necessary to install 2104 re-bar rockbolts which are 2.22 cm (7/8 in) in diameter and 3.7 m (12 ft) long to support the surface of the cavity and provide a means of attaching the wire mesh. These rockbolts are installed on a 1.52 x 1.52 m (5 x 5 ft) pattern. The total drill length is 50,800 m (166,800 ft) and include borehole sizes from 4.45 to 7.62 cm (1-3/4 to 3 in).

5.2.6 73.2 Meter (240 ft) External Support Cavity

This cavity design represents the first major step beyond what can be regarded as current practice in cavern excavation. Since the rock support and excavation methods have been previously employed in other excavation projects, it is not considered beyond the state-of-the-art. This cavern represents an excavated volume 8 times that of the Red Hot and Deep Well cavities and it is expected that several construction techniques such as accurate long hole drilling and reliable tendon bearing surface anchorage would need to be perfected. The layout of the rock support system is shown in Figure 5-7 and includes 3 annular tendon galleries in order to meet the tendon/cavity surface intersection angle requirements. The rock load depth is 14.6 m·(48 ft) and the annular drifts are again located at twice this distance from the cavity surface. The design support capacity is 711,700 Newtons (160 kips) and tendon and Dywidág rock anchors are specified with load capacities of 771,800 and 738,000 Newtons (173.9 and 165.9 kips) respectively. A total of 2956 tendons and 282 rockbolts are used to support the cavity crown and sidewalls, 748 Dywidag rockbolts are required to pin the floor, and 3739 re-bar rockbolts are necessary to support the surface of the cavity. The total drill hole length is 115,900 m (380,200 ft) and again includes borehole sizes from 4.45 to 7.62 cm (1-3/4 to 3 in).

5.2.7 91.4 Meter (300 ft) External Support Cavity

This cavity is the largest evaluated in this report since it represents such a great extension of the level of current practice in cavity excavation. The excavated volume of this cavity would be nearly 16 times that previously mined in Rainier Mesa and between 6 and 8 times that of any previous cavern excavation. The mining of a chamber of this size appears to be within the capabilities of rock support system design and the mining methods. However, as the chamber size increases, the probability of encountering joints and bedding plane weaknesses also increases, and therefore installation of additional support to restrict movement of unstable rock formations may be necessary. The rock support layout is shown in Figure 5-8 and is very similar to that for the 73.2 m (240 ft) cavity employing 3 annular tendon galleries. The rock load depth is 18.3 m (60 ft) and the tendons range in length from 36.6 to 40.8 m

(120 to 134 ft). The design support capacity is 889,600 Newtons (200 kips) and was the basis for the 1.63 m (5.4 ft) support spacing used for all cavity designs. Tendons with a working capacity (60 percent of ultimate) of 881,600 Newtons (198.2 kips) and Dywidag rockbolts with a lockoff capacity of 738,000 Newtons (165.9 kips) are specified for the cavity support. A total of 4675 tendons and 352 rockbolts are used to support the crown and sidewalls, 1149 Dywidag rockbolts are required to pin the floor and corner, and 5842 re-bar rockbolts are needed to support the cavity surface and wire mesh. Boreholes for the rockbolts range in size from 4.45 to 8.89 cm (1-3/4 to 3-1/2 in) and require a total drill length of 218,000 m (715,150 ft).

5.3 ALTERNATE CAVITY DESIGN

The evaluation of the cost and feasibility of large cavity construction as presented in this report only considers designs in which the caverns are either completely supported internally or completely supported from external galleries. This was done in order to provide a consistent basis for cost estimation and so that the effect of important parameters could be clearly seen. However, it is felt that for cavities larger than 45.7 m (150 ft) in diameter a combination of internal and external support could prove to be the optimum design. Such a combination of support would involve the use of external galleries near the crown of the cavity in order to provide a means of pre-supporting the rock mass forming the chamber roof and to allow for constant monitoring of the behavior of this critical region. Internal support would be employed in the less critical sidewall region and for pinning the floor and corner of the cavern. A possible layout combining the use of internal and external support is shown in Figure 5-9 for a 54.9 m (180 ft) cavity. From a cost standpoint, such a design significantly reduces the development drift and rockbolt drill footage, allows for efficient mining in the confined area near the roof of the cavity, and permits the use of the less expensive rockbolts in the lower regions of the cavity where more men and equipment can be efficiently used. It is expected that optimization of the rock support system and the mining methods would be part of the final design effort for any planned excavation.

TABLE 5-1. DIMENSIONAL DATA FOR THE MODIFIED HEMISPHERICAL CAVITIES

Cavity Span (D)	80	120	160	180	240	300
Cavity Height (P)	45	68	91	102	136	170
Spherical Zone (H_1)	28	42	57	64	85	106
Paraboloridal Zone (H_2)	17	26	34	38	51	64
Paraboloid Length (L)	57	85	113	127	170	212
Cavity Arc Length (S)	130	195	260	293	390	488
Spherical Zone (S_1)	63	94	126	141	189	236
Paraboloidal Zone (S_2)	67	101	135	151	202	252
Cavity Surface Area (A)	10,400	23,400	41,500	52,600	93,500	146,100
Spherical Zone (A_1)	7,100	16,000	28,400	36,000	64,000	100,000
Paraboloidal Zone (A_2)	3,300	7,400	13,100	16,600	29,500	48,100
Cavity Volume (V)	140,000	470,000	1,120,000	1,590,000	3,780,000	7,380,000
Spherical Zone (V_1)	120,000	400,000	950,000	1,350,000	3,200,000	6,250,000
Paraboloidal Zone (V_2)	21,000	72,000	170,000	240,000	580,000	1,130,000
Depth of Cavity Influence (.2D)	16	24	32	36	48	60
Span Ratio (D/D_{300})	.267	.400	.533	.600	.800	1.000
Area Ratio (A/A_{300})	.071	.160	.284	.360	.640	1.000
Volume Ratio (V/V_{300})	.019	.064	.152	.216	.512	1.000

NOTE: Dimensions in this table are in ft, ft^2, and ft^3

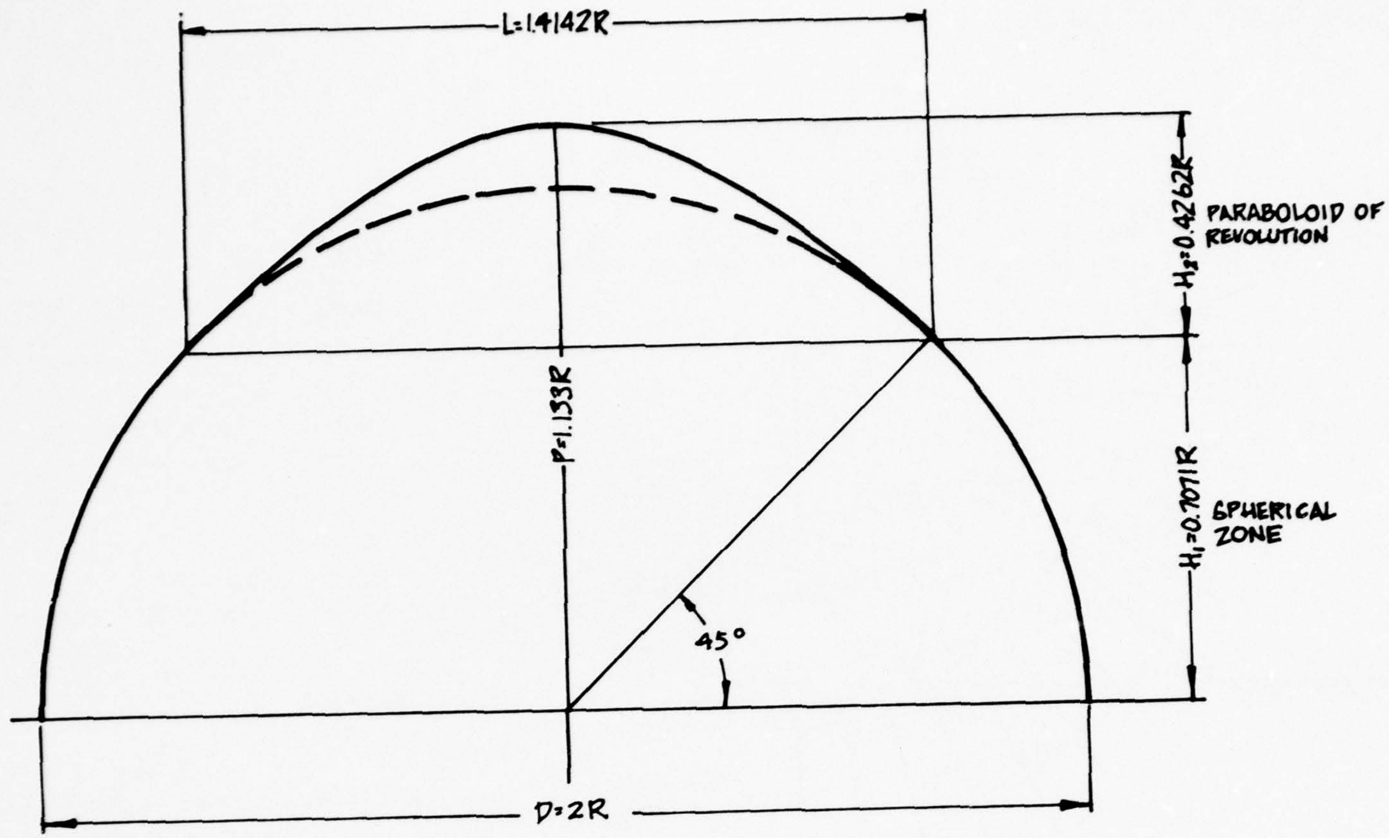


FIGURE 5-1. PRELIMINARY LARGE CAVITY DESIGN SHAPE

Zone of Influence = 16 ft
Rock Load = 116 x 16 = 1856 lbs/ft²

Use Re-bar Rockbolts
Bar Diameter = 1 1/8"
Ultimate Strength = 80k
Yield Strength = 60K (Design = 53.3 K)
Drill Hole = 1 3/4" ϕ

		Drill Ft.
Paraboloidal Zone		
60 Re-bar RB's	1 1/8 ϕ x 28'	1680
59 Re-bar RB's	1 1/8 ϕ x 22'	1298
Spherical Zone		
119 Rebar RB's	1 1/8 ϕ x 22'	2618
138 Re-bar RB's	1 1/8 ϕ x 16'	2208
Hemispherical Base		
157 Re-bar RB's	1 1/8 ϕ x 16'	2512
		10,316

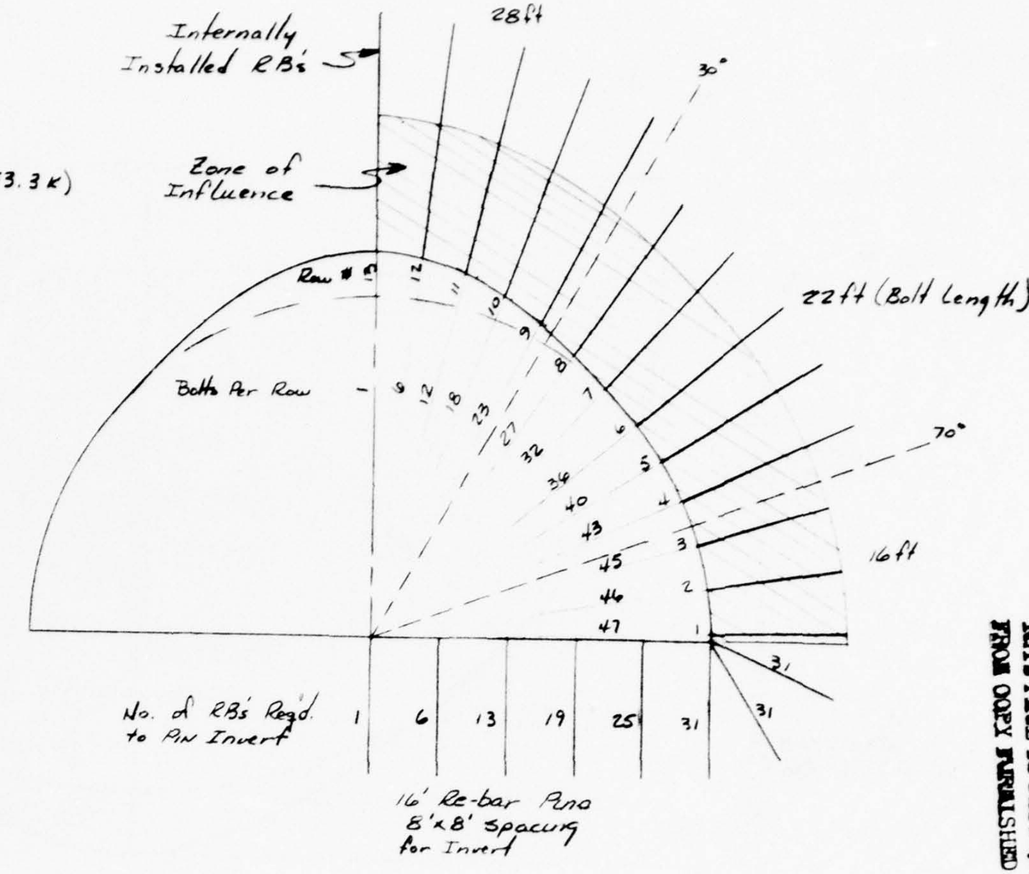


FIGURE 5-2. 24.4 m (80 ft) INTERNAL SUPPORT CAVITY

Zone of Influence = 24ft
Rock Load = 116 × 24 = 2784 lbs/ft²

Use Mono-bar Dywidag Rock Anchors
Bar Diameter = 1"
Ultimate Strength = 127.5K
Lockoff Strength = 89.3K (Design = 80K)
Drill Hole = 2 3/8" ϕ

			Drill Ft.
Paraboloidal Zone			
124	Dywidag RB's	1ϕ × 40'	4,960
130	Dywidag RB's	1ϕ × 32'	4,160
Spherical Zone			
296	Dywidag RB's	1ϕ × 32'	9,472
276	Dywidag RB's	1ϕ × 24'	6,624
Hemispherical Base			
175	Dywidag RB's	1ϕ × 24'	4,200
95	Dywidag RB's	1ϕ × 20'	1,900
			31,316

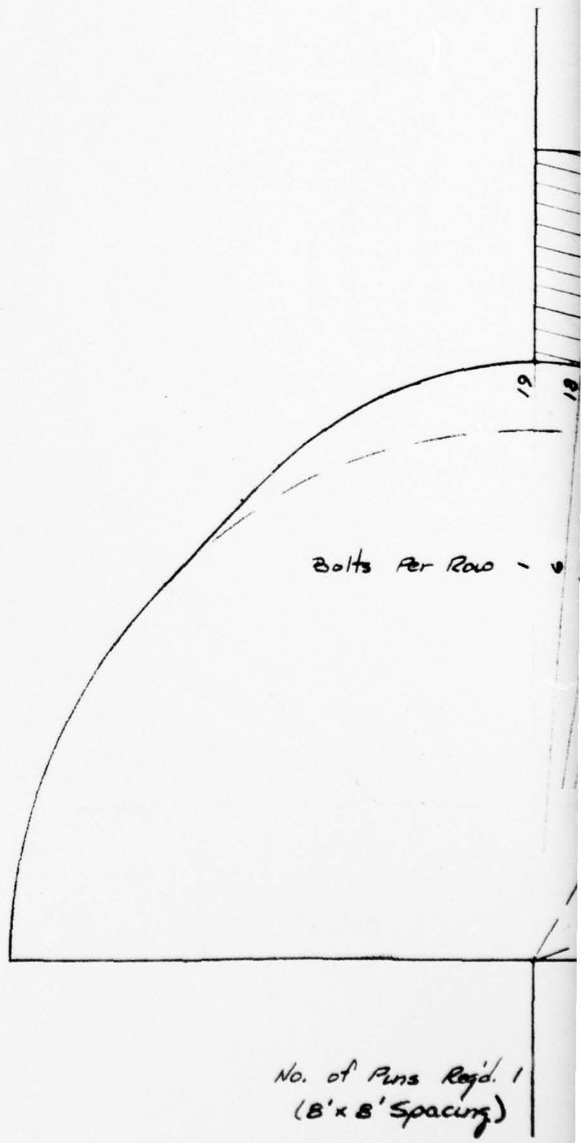
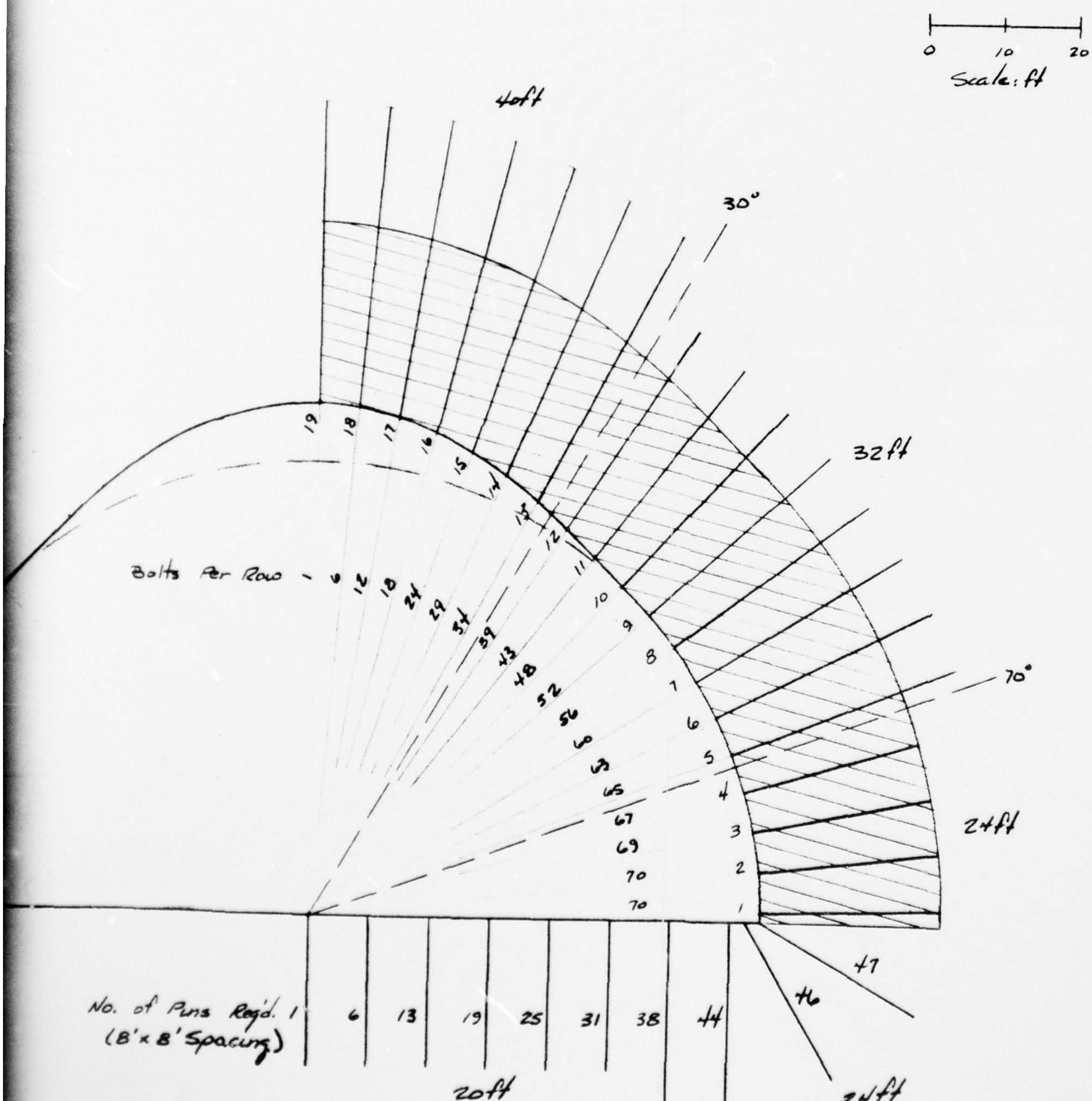


FIGURE 5-3. 36.6 m (120 ft) INTERNAL SUPPORT CAVITY


0
10
20
Scale: ft
40ft
30°
32ft
70°
24ft
Bolts Per Row - 6 12 18 24 29 34 39 43 48 52 56 60 63 65 67 69 70 70
19 18 17 16 15 14 13 12 11 10 9 8 7 6 5 4 3 2 1
No. of Pins Req'd. 1 6 13 19 25 31 38 44
(8'×8' Spacing)
20ft
47
46
24ft

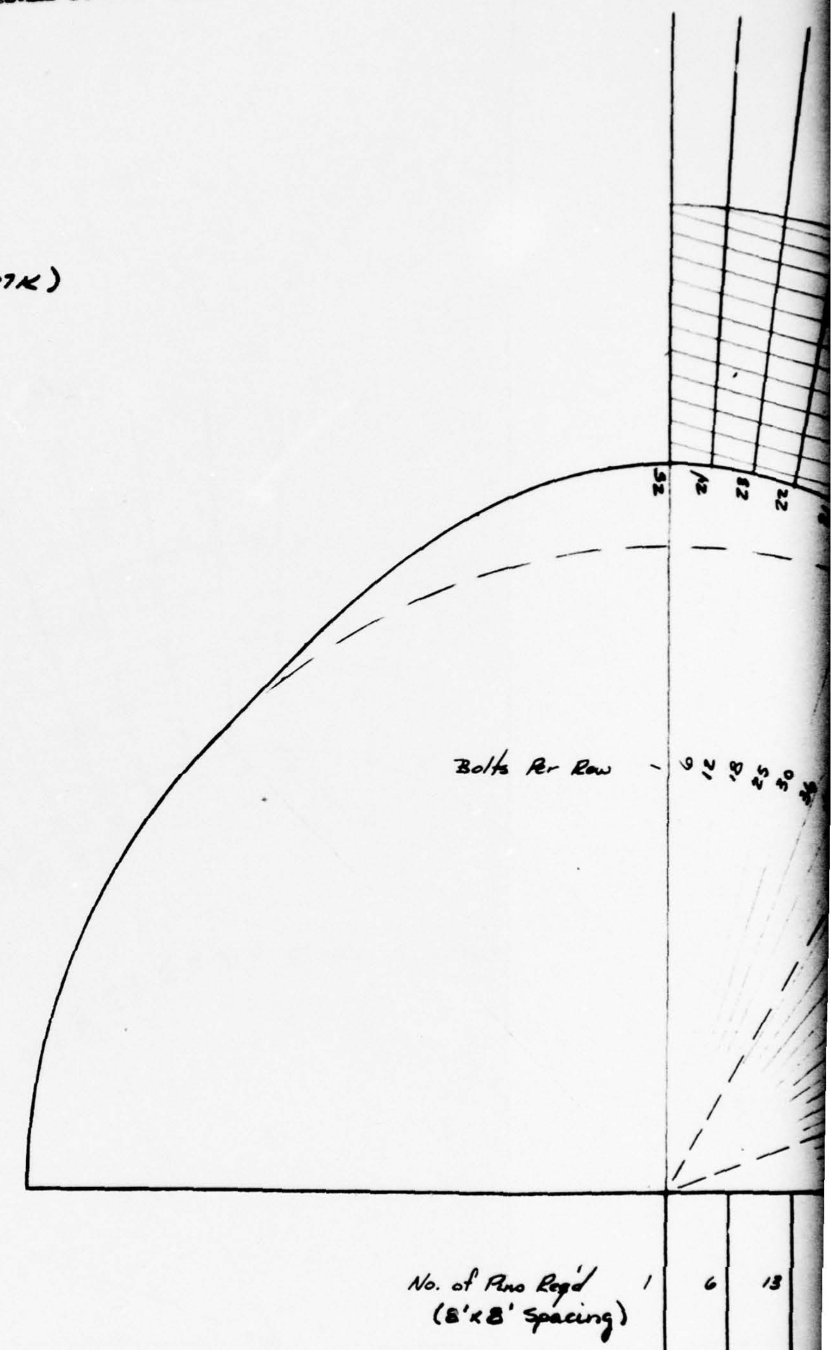
AD-A067 218 ENGINEERING DECISION ANALYSIS CO INC IRVINE CA F/G 8/9
COST AND FEASIBILITY EVALUATION FOR THE EXCAVATION OF LARGE HEM--ETC(U)
OCT 78 T R KIPP, R P KENNEDY DNA001-78-C-0281
UNCLASSIFIED EDAC-177-041.1R DNA-4723T NL

2 OF 3
AD A067218

Zone of Influence = 32 ft
Rock Load = 116 × 32 = 3712 lbs/ft²

Use Mono-bar Dywidag Rock Anchors
Bar Diameter = 1¼"
Ultimate Strength = 187.5 K
Lock off Strength = 131.3 K (Design = 107 K)
Drill Hole = 2 9/16" ϕ

			Drill Ft.
Paraboloidal Zone			
215	Dywidag RB's	1¼ϕ × 56'	12,040
230	Dywidag RB's	1¼ϕ × 44'	10,120
Spherical Zone			
465	Dywidag RB's	1¼ϕ × 44'	20,460
550	Dywidag RB's	1¼ϕ × 32'	17,600
Hemispherical Base			
350	Dywidag RB's	1¼ϕ × 32'	11,200
133	Dywidag RB's	1¼ϕ × 20'	2660
			74,080



FIGURE

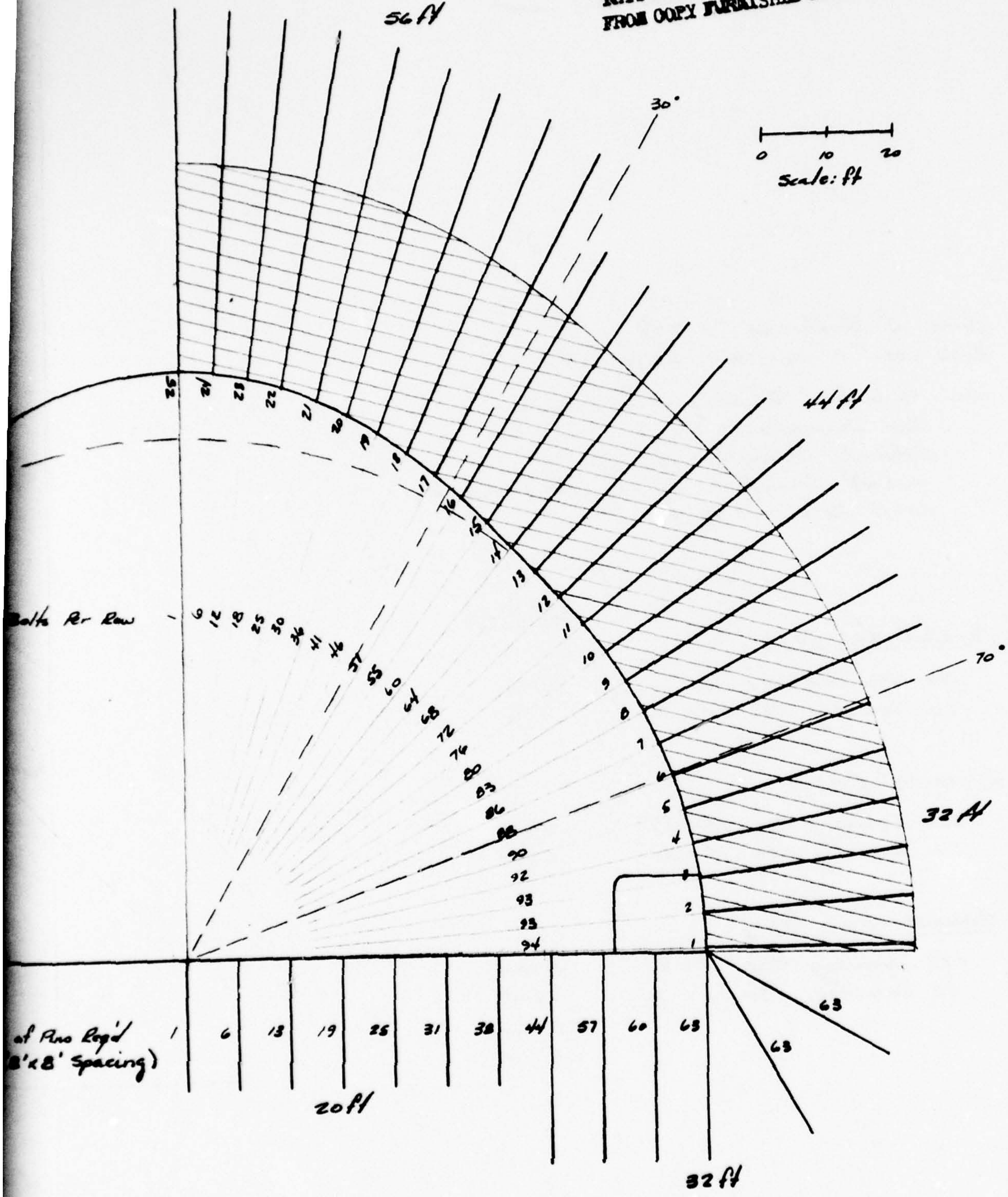
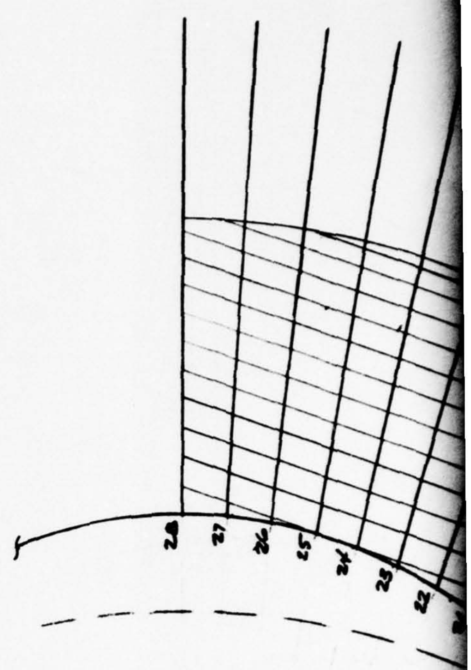


FIGURE 5-4. 48.8 m (160 ft) INTERNAL SUPPORT CAVITY

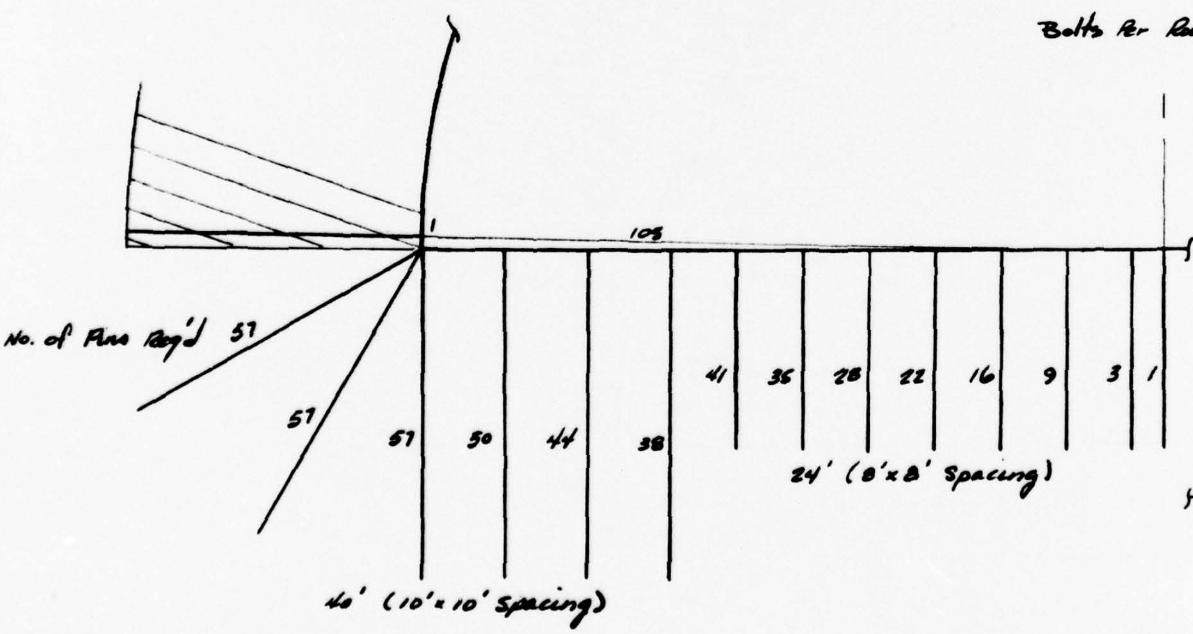
2

Zone of Influence = 36 ft
Rock Load = 116 × 36 = 4176 lbs/ft²

Use Mono-bar Dywidag Rock Anchors
Bar Diameter = 1¼"
Ultimate Strength = 187.5K
Lockoff Strength = 131.3K (Design = 120K)
Drill Hole = 2 9/16" ϕ



			Drill Ft
Paraboloidal Zone			
265	Dywidag RB's	1¼ϕ × 60'	15,900
327	Dywidag RB's	1¼ϕ × 48'	15,696
Spherical Zone			
623	Dywidag RB's	1¼ϕ × 48'	29,904
619	Dywidag RB's	1¼ϕ × 36'	22,284
Hemispherical Base			
303	Dywidag RB's	1¼ϕ × 40'	12,120
155	Dywidag RB's	1¼ϕ × 24'	3,720
			99,624



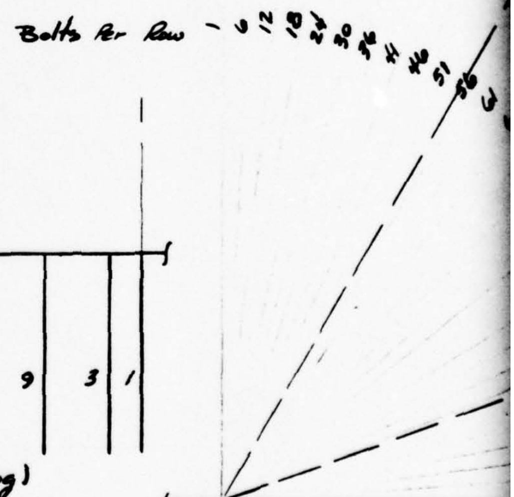
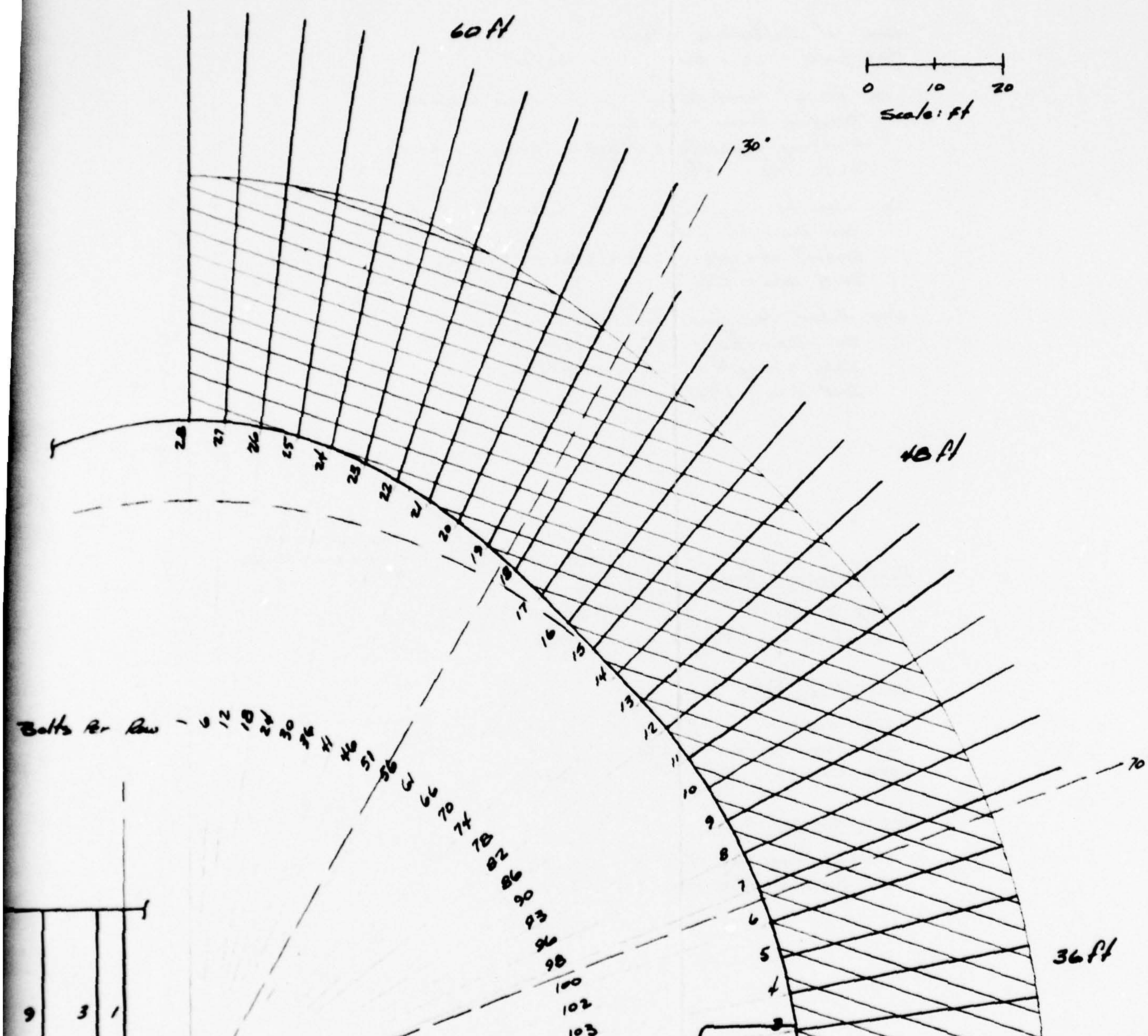


FIGURE 5-5. 54.9 m (180 ft) INTERNAL SUPPORT CAVITY


60 ft
0 10 20
Scale: ft
30°
48 ft
Bolts Per Row
70°
36 ft
28 27 26 25 24 23 22 21 20 19 18 17 16 15 14 13 12 11 10 9 8 7 6 5 4 3 2 1
9 3 1

Zone of Influence = 36 ft
Rock Load = 116 × 36 = 4176 lbs/ft²

Use V.S.L. Prestressed Tendon Rock Anchors
Tendon Area = 0.765 in²
Working Capacity = 123.9K (Design = 120K)
Drill Hole = 3" ϕ

Use Mono-bar Dywidag Rock Anchors
Bar Diameter = 1¼"
Lockoff Strength = 131.3 K (Design = 120 K)
Drill Hole = 2 9/16" ϕ

Use Rebar Personnel Rockbolts
Bar Diameter = 7/8"
Yield Strength = 26K (Design = 25K)
Drill Hole = 1¾" ϕ

	Drill Ft.
Paraboloidal Zone	
592 Tendons	44,950
664 Personnel RB's	7,968
Spherical Zone	
928 Tendons	69,461
314 Dywidag RB's 1¼ϕ × 36'	11,304
1440 Personnel RB's	17,280
Hemispherical Base	
303 Dywidag RB's 1¼ϕ × 40'	12,120
155 Dywidag RB's 1¼ϕ × 24'	3,720
	166,803

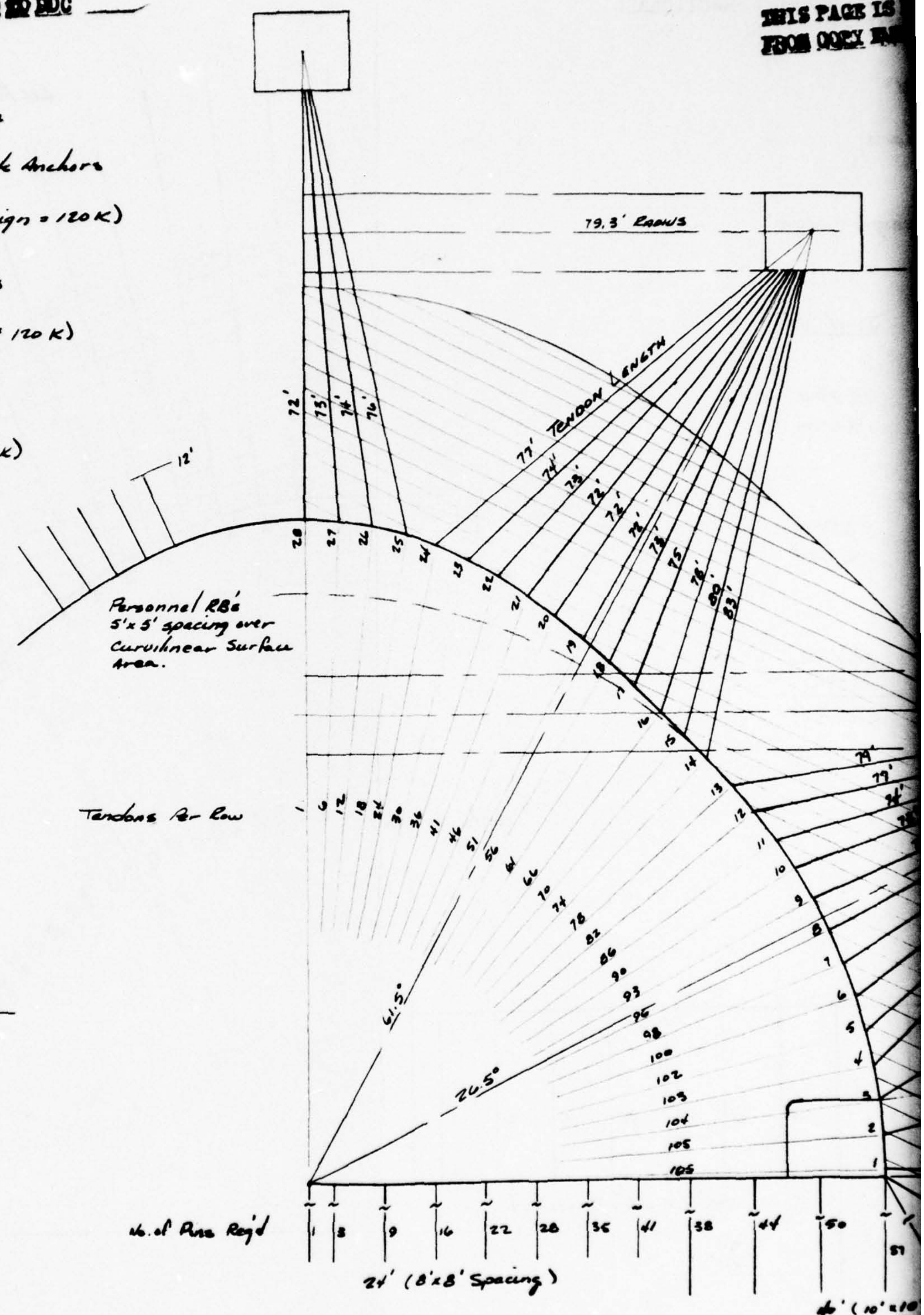


FIGURE 5-6. 54.9 m (180

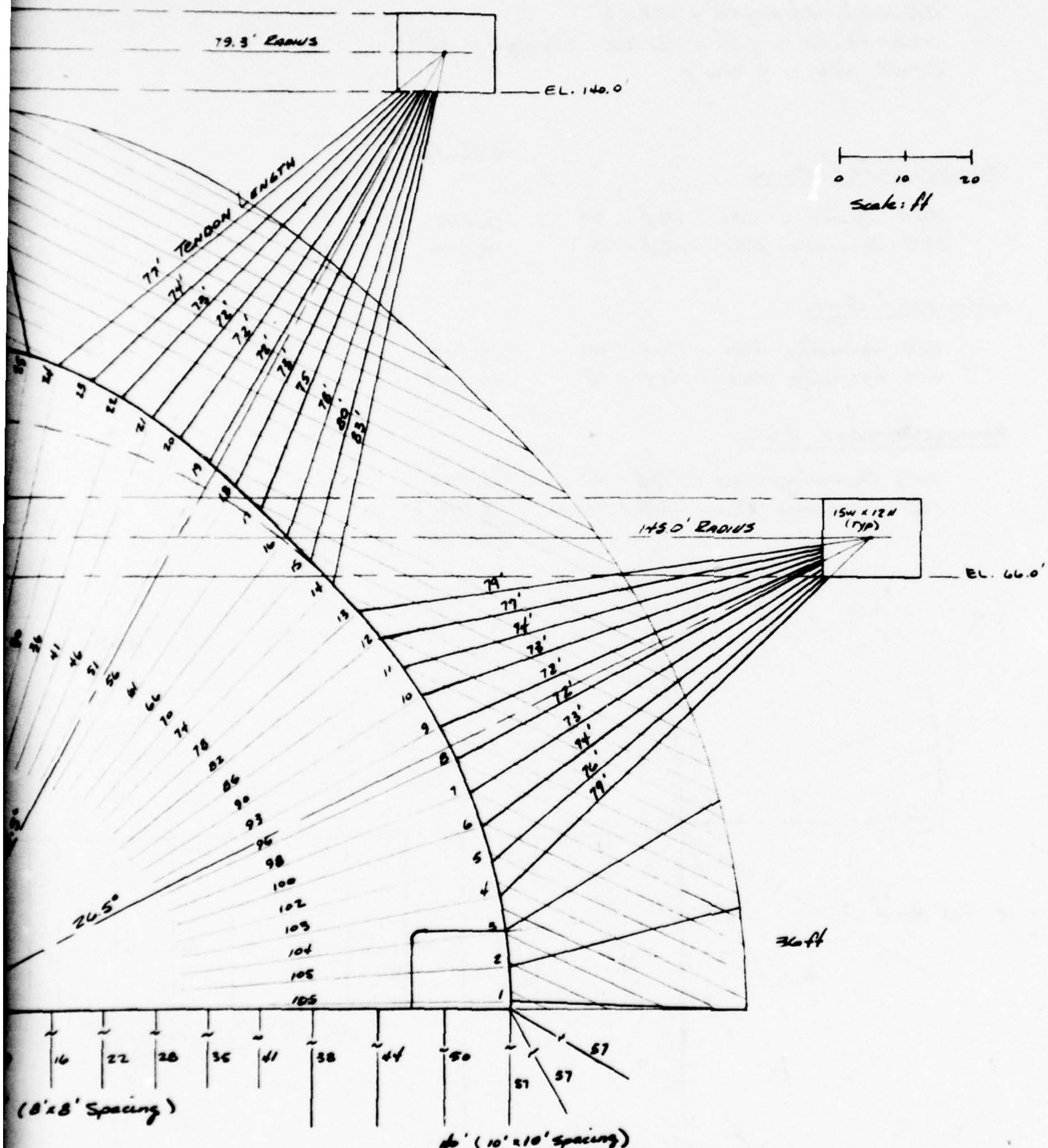


FIGURE 5-6. 54.9 m (180 ft) EXTERNAL SUPPORT CAVITY

4

Zone of Influence = 48 ft
Rock Load = 116 × 48 = 5568 lbs/ft²

Use V.S.L. Prestressed Tendon Rock Anchors
- Tendon Area = 1.071 in²
- Working Capacity = 173.5 K (Design = 160 K)
- Drill Hole = 3" ϕ

Use Mono-bar Dywidag Rock Anchors
- Bar Diameter = 1 3/8"
- Lockoff Strength = 165.9 K (Design = 160 K)
- Drill Hole = 2 3/4" ϕ

Use Rebar Personnel/Rockbolts
- Bar Diameter = 7/8"
- Yield Strength = 36 K (Design = 25 K)
- Drill Hole = 1 1/4" ϕ

	Drill ft.
Paraboloidal Zone	
1063 Tendons	105,764
1180 Personnel RB's	14,160
Spherical Zone	
1893 Tendons	184,287
282 Dywidag RB's 1 3/8" ϕ × 48'	13,536
2559 Personnel RB's	30,708
Hemispherical Base	
464 Dywidag RB's 1 3/8" ϕ × 50'	23,200
284 Dywidag RB's 1 3/8" ϕ × 30'	8520
	380,175

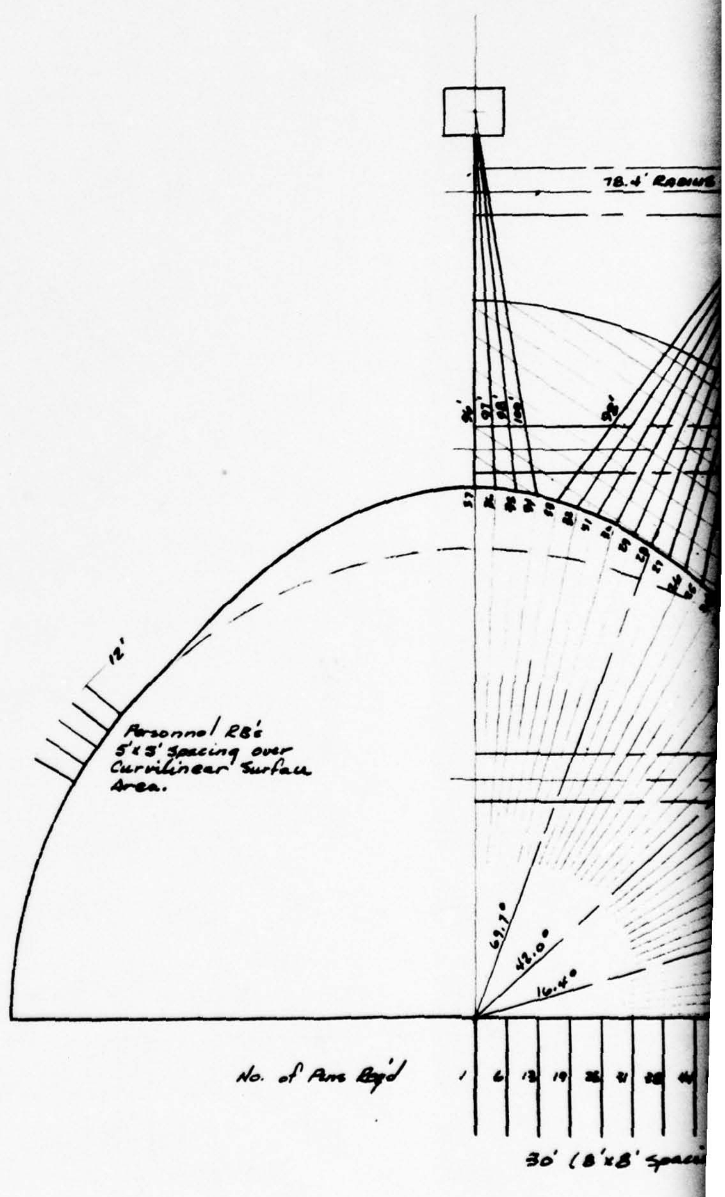


FIGURE 5-7. 73.2 m (240 ft) EXTERNAL SUPPORT CAVITY

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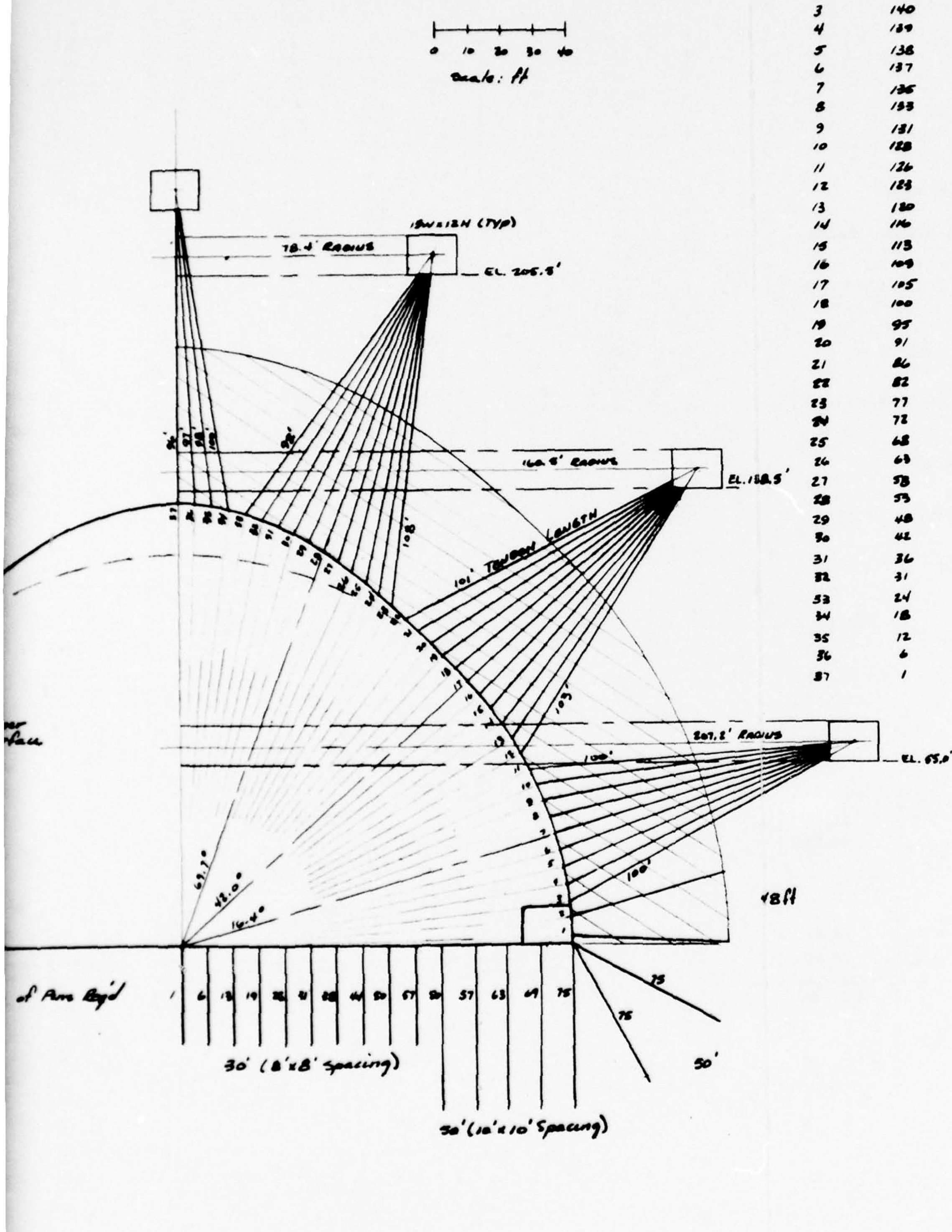

TENDON REQUIREMENTS

Row	Number	Length
1	141	48
2	141	48
3	140	100
4	139	99
5	138	98
6	137	97
7	136	96
8	133	97
9	131	98
10	128	99
11	126	100
12	123	103
13	120	100
14	116	98
15	113	97
16	109	96
17	105	96
18	100	96
19	95	97
20	91	98
21	86	99
22	82	101
23	77	108
24	72	105
25	68	103
26	63	101
27	58	99
28	53	97
29	48	96
30	42	96
31	36	96
32	31	97
33	24	98
34	18	100
35	12	98
36	6	97
37	1	96

2

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TENDON REQUIREMENTS

Row	Number	Length	Row	Number	Length
1	176	60	24	116	124
2	176	60	25	112	134
3	175	124	26	107	131
4	174	123	27	103	129
5	173	122	28	99	127
6	172	121	29	94	125
7	171	120	30	90	123
8	170	121	31	85	122
9	168	122	32	80	121
10	166	123	33	75	120
11	164	124	34	69	120
12	161	128	35	64	121
13	159	128	36	59	122
14	156	126	37	54	123
15	153	124	38	48	125
16	149	123	39	42	127
17	146	122	40	36	129
18	142	120	41	30	128
19	139	120	42	24	124
20	135	120	43	18	122
21	131	121	44	12	121
22	126	122	45	6	120
23	121	123	46	1	120

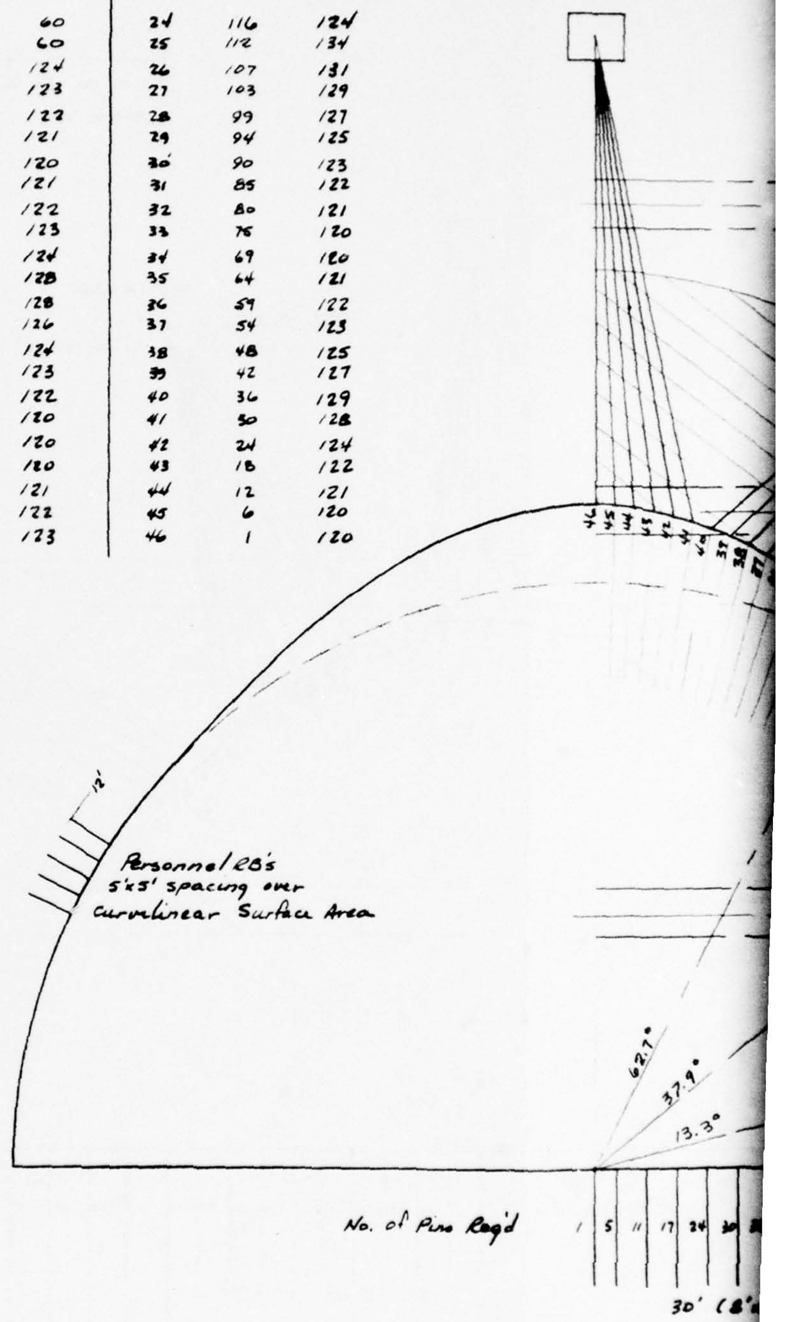
Zone of Influence = 60 ft
Rock Load = 116 × 60 = 6960 lbs/ft²

Use V.S.L. Prestressed Tendon Rock Anchors
Tendon Area = 1.224 in²
Working Capacity = 198.2K (Design = 200K)
Drill Hole = 3½" ϕ

Use Mono-bar Dywidag Rock Anchors
Bar Diameter = 1⅜"
Lockoff Strength = 165.9K (Design = 160K)
Drill Hole = 2¾" ϕ

Use Rebar Personnel Rockbolts
Bar Diameter = ⅞"
Yield Strength = 26K (Design = 25K)
Drill Hole = 1¾" ϕ

		Drill Ft.
Paraboloidal Zone		
1545	Tendons	193,168
1843	Personnel RB's	22,116
Spherical Zone		
3130	Tendons	384,369
352	Dywidag RB's 1⅜"ϕ × 60'	21,120
3999	Personnel RB's	47,988
Hemispherical Base		
596	Dywidag RB's 1⅜"ϕ × 50'	29,800
553	Dywidag RB's 1⅜"ϕ × 30'	16,590
		715,151



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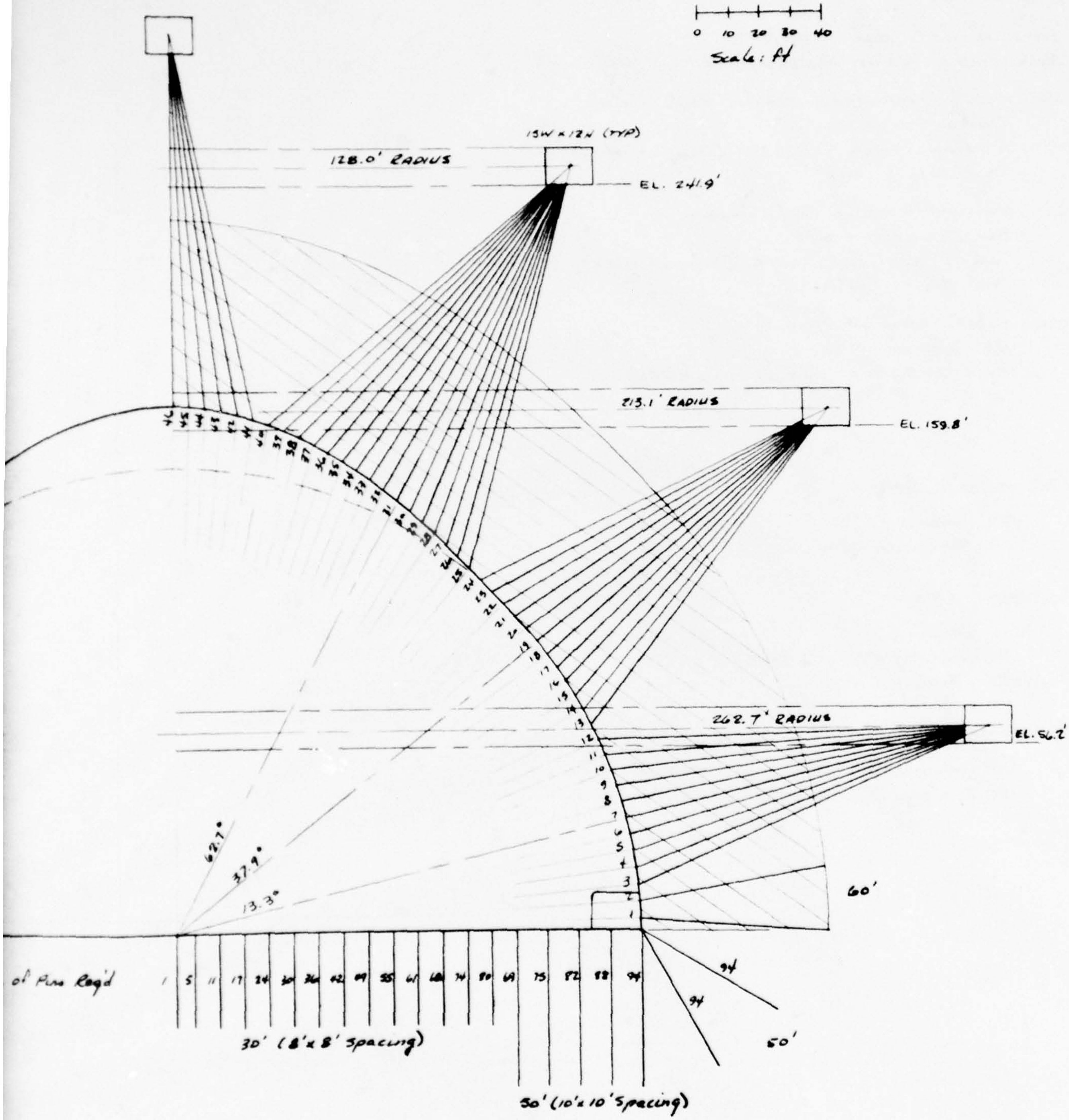


FIGURE 5-8. 91.4 m (300 ft) EXTERNAL SUPPORT CAVITY

2

			Drill Ft
Paraboloidal Zone			
592	Tendons		44,950
664	Personnel RB's		7,968
Spherical Zone			
1242	Dywidag RB's	1¼φ × 36'	44,712
Hemispherical Base			
303	Dywidag RB's	1¼φ × 40'	12,120
155	Dywidag RB's	1¼φ × 24'	3,720
			113,470

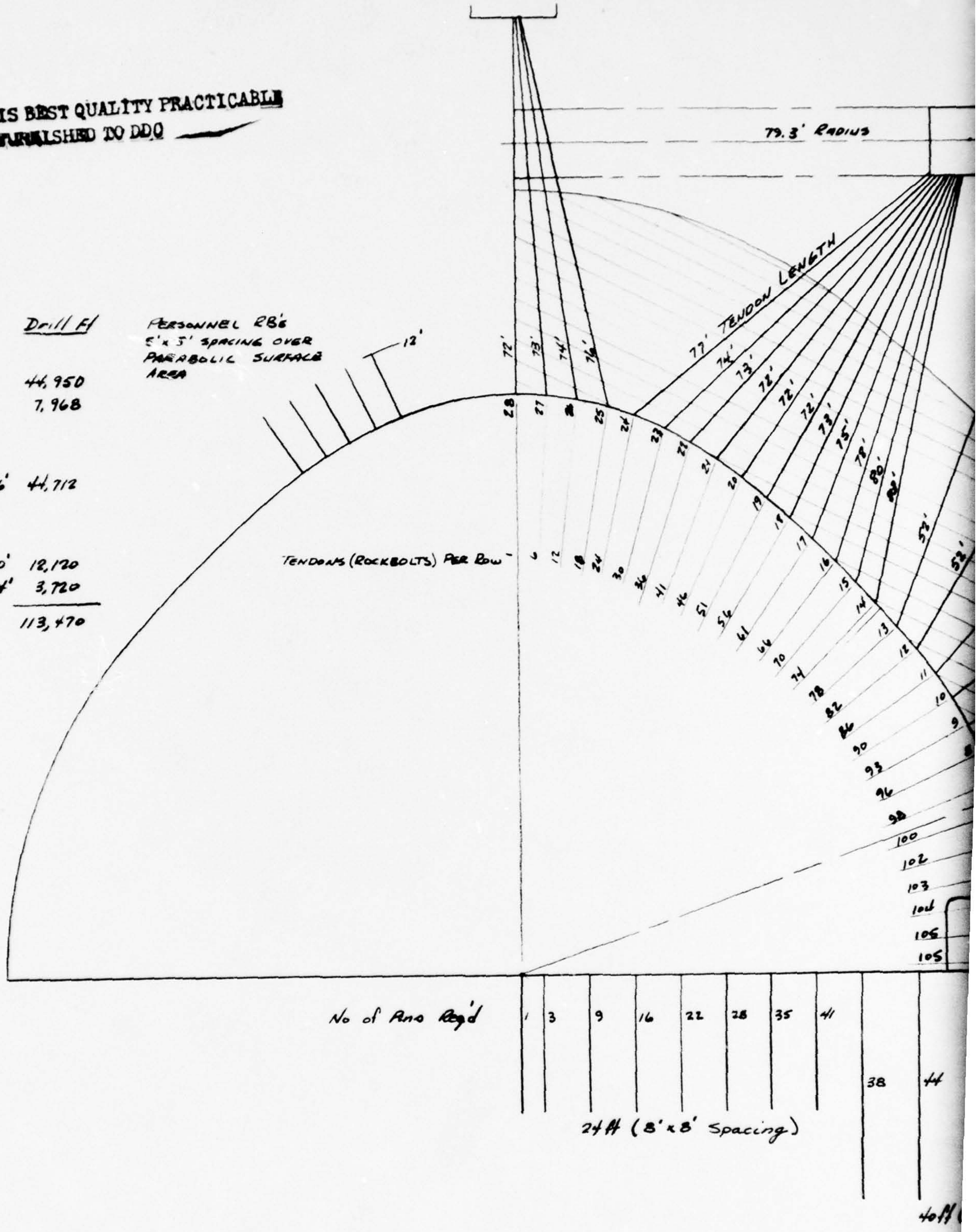
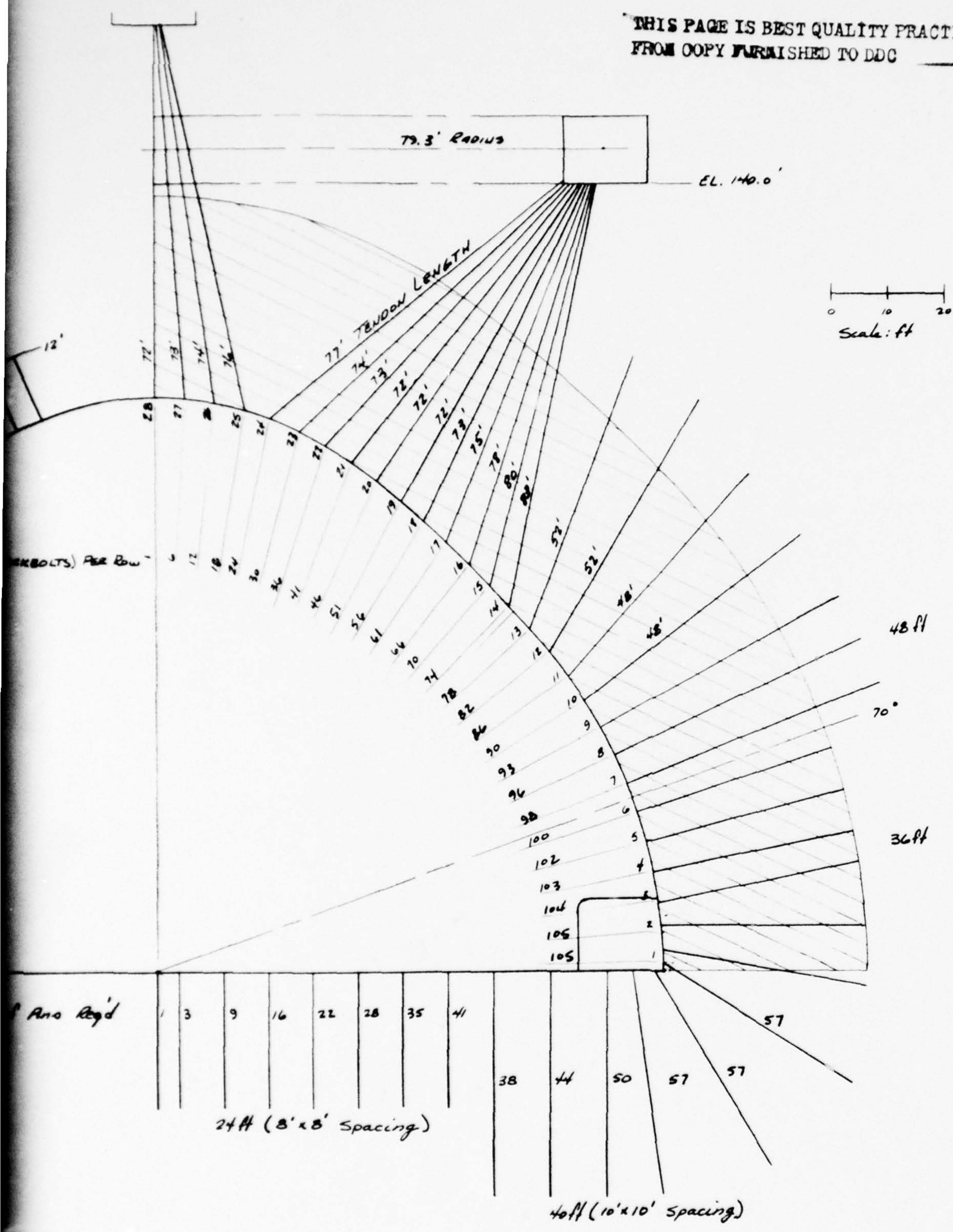


FIGURE 5-9. 54.9 m (180 ft) COMBINED INTERNAL/EXTERNAL SUPPORT CAVITY



...ERNAL SUPPORT CAVITY

6. ADDITIONAL ANALYSES REQUIRED FOR FINAL CAVITY DESIGN

The technical basis for the cost and feasibility evaluation of the NTS large underground cavity contained in this report has been dependent upon inferred geologic conditions and upon design parameters determined from past experience. Since the geology and material property values have been inferred from actual data taken in the near vicinity of the proposed site, and since the design parameters are highly influenced by experience gained in the excavation of the Red Hot and Deep Well cavities in Rainier Mesa, it is expected that the bases for design very nearly represent the conditions at the proposed site. However, since this evaluation program is generic in nature, an accurate geologic characterization of the final selected site is important in order to provide a proper technical basis for final cavity design. A number of geology, laboratory, and engineering analyses coupled with some field construction experiments are recommended as part of the future design effort necessary to finalize the cavity shape and support requirements.

6.1 CORE HOLES AND EXPLORATORY DRIFTS

Core holes and exploratory drifts are the source of data for determining material properties and regional lithology. It is expected that the selection or final approval of an excavation site for the large cavity will be based upon core hole data and that the site selection process will contain sufficient flexibility to accomodate some adjustment in the specific cavern location, based upon the results of the initial boring program. As a minimum three exploratory core holes should be drilled. One horizontal hole should be drilled perpendicular to the primary joint or fault system which is thought to be striking NW at the proposed cavern site; a second horizontal hole should be drilled perpendicular to the first (parallel to the fault system) to ensure that no other major geological structures are encountered; and thirdly, a vertical hole should be drilled and cored at least 45.7 m (150 ft) above and 22.9 m (75 ft) below the cavern interval. In addition, during the excavation

of the initial development drifts and raises, the geology should be mapped and extrapolated to the chamber site in order to determine the expected rock conditions.

6.2 LITHOLOGY

The mapping and interpretation of those geologic features which will influence cavern stability and support requirements are the most important pre-construction tests that can be conducted. The location and thickness of weak friable zones should be established together with the location and thickness of closely spaced bedding plane partings. The location of near vertical joints, fractures, and fault zones must also be accurately established. Faults or joints striking in the east-west direction are expected to be less frequent than the north-south trending faults. However, some may exist and the exploration program should determine their location. It may be advantageous to obtain oriented core samples in order to determine the strike of joint systems. The vertical drill hole should provide information pertaining to clay beds, silicified layers, bedding planes and other discontinuities while the initial drifts in the area will provide a more complete picture of the joint and fault orientations.

6.3 LABORATORY TESTS

As discussed briefly in Sections 2 and 3 and described in detail in Reference 13, comparing the material property values of the in-situ rock mass, including regions of discontinuity, to the property values of the intact portion of the rock mass provides a means of assessing the quality of the rock mass by means of the Rock Quality Designation (RQD) index. Indexing of the rock in the cavern area, especially for the region between 45.7 or (150 ft) above and 22.9 m (75 ft) below the cavity interval, should be accomplished and requires that a number of laboratory tests be conducted on the core samples obtained from the site. Unconfined compression tests should be performed on natural water content specimens of all significant horizons in the tuff. The unconfined compressive strength of the rock with respect to the in-situ stresses can be used as an index of the degree of new fracturing that is expected

to develop during excavation. The deformation modulus, E, of the compression test sample represents an upper limit for the stiffness of the rock mass. Sonic compressional velocity, density, and water content measurements provide a further means of indexing of the core. The number of tests required would depend on the subject lithology but should be adequate to define any variation between the tunnel bed sub-units and material variation within the sub-units.

Triaxial compression and extension tests should be performed on the tuff to determine its confined strength and deformability. The information will also assist in establishing the depth of fracturing expected. If the subject area contains significant faulting, a few direct shear tests would be appropriate to define the frictional properties.

6.4 FIELD TESTS

Compressional velocities should be obtained in the horizontal or vertical bore holes penetrating the proposed site. Since sonic velocities are lower in rock which is fractured or contains gas voids, the comparing of these data to the compressional velocity values obtained from intact core samples provides a good correlation to rock quality.

Evaluation of the in-situ state of stress should include at least one stress determination using the U.S. Bureau of Mines overcore method. However, since it is not expected that the feasibility or location of cavern construction will be dependent upon additional stress data, measurements could be obtained during the excavation of the initial access drifts. Once access tunnels to the cavity area have been mined, one or more additional stress determinations could be conducted to verify the uniformity of the stress field thoughout the excavation site. If preliminary data is desired, some data concerning stress conditions could be obtained by hydrofracturing techniques in drill holes used for geologic site exploration and geophysical measurements.

6.5 ENGINEERING ANALYSES

Although much of the design of the cavity and the rock support systems is based upon past experience and the monitoring of rock behavior as excavation

progresses, significant information concerning minimum rock displacements and new fracturing around the opening can be obtained from engineering analysis techniques.

Minimum rock displacements can be estimated using an elastic analysis. Extensometer movements during excavation which are large with respect to the predicted elastic movement, may be indicative of developing unstable conditions.

The new fracturing that develops around the opening can be evaluated using an elasto-plastic stress analysis. An axisymmetric finite element program can be used as a first step in analyzing both elastic and elasto-plastic behavior of the cavern. Appendix B presents a closed-form solution for determining the radius of the plastic zone around a sphere, when it is assumed that vertical and horizontal stresses are equal.

Analyses should also be carried out using a joint element, elasto-plastic finite element model to determine the combined effect of the low rock strength and the presence of vertical joints and bedding plane partings on the formation of large wedges that require support. Useful results can be obtained with an axisymmetric program. Although a true three-dimensional program might provide a more complete evaluation, there is some question, at present, as to the ability to obtain meaningful results with a full three-dimensional program because of its complexity. The results of these analyses should be correlated with past performance in the Red Hot/Deep Well caverns.

The support required in the cavern can be realistically and conservatively determined by evaluating the potentially unstable rock wedges requiring support. The geometry of the major joint and bedding plane systems, as determined from the borehole data discussed above, can be used to establish wedge sizes. The support recommendations outlined in this report were made on the basis of such a procedure, relying heavily on the observed behavior in the Red Hot/Deep Well caverns.

6.6 CONSTRUCTION TESTS

Since the construction of the larger cavities evaluated in this report will involve rock support systems which must span longer lengths and carry higher rock loadings than in previously excavated caverns, and since only limited experience has been gained in anchoring rock support systems in external tendon galleries, it is recommended that construction tests be conducted to evaluate or develop the following:

1. Procedures for tendon anchorage.
2. Procedures for long hole drilling alignment and accuracy.
3. Bearing pads suitable for use in the cavern or the external galleries.
4. Techniques for long hole instrumentation. Extensometers should be tested in boreholes of similar length to those proposed for the cavern construction.

7. CONSTRUCTION PROCEDURES AND ESTIMATES OF COST AND SCHEDULE

This section describes the design considerations, mining methods, and assumptions which formed the bases for estimates of cost and schedule pertaining to the construction of the underground chambers. It was the intent of this evaluation to present a consistent basis for estimation so that important cost parameters such as variation with size and the effect of internally versus externally supported designs can be clearly seen. As a result the support design, mining methods, and construction sequences were not optimized for each individual cavity size. An attempt was made to establish the optimum method of mining and construction for the 48.8 m (160 ft) diameter internally supported cavern configuration. The optimized costs and schedule are also included in this section and are compared to the consistent basis estimates.

Section 5 of this report presented the design details for each cavity size and forms the basis for shape and rock support considerations. For purposes of estimation, either completely internal or completely external rock support was assumed for the appropriate cavities and no attempt was made to optimize the combined use of externally installed tendons and internally installed primary rockbolts in the wall or crown of the chambers. In order that the formation of new fractures near the surface of the cavities might be minimized, the cost and schedule estimates only consider the use of Alpine miners for chamber excavation within a distance of approximately 6 m (20 ft) of the surface. Due to the desire to employ mechanical miners near the surface of the cavity, it is clear that it would be most efficient to use them to complete the excavation of the smaller caverns. Similarly it is clear that several of the larger chambers could be efficiently constructed combining the use of Alpine miners and conventional techniques. However, in order to provide a consistent basis for estimation, it was assumed that the entire cavern excavation for each cavity would be limited to the use of Alpine miners and that conventional drill and blast procedures would not be considered. Conventional drill and blast pro-

cedures for the removal of the core of the larger caverns should be considered in later design stages.

7.1 INTERNALLY SUPPORTED CAVITY CONSTRUCTION

As discussed in Section 5, an internally supported cavity design was considered for cavities which are 24.4, 36.6, 48.8, and 54.9 m (80, 120, 160, and 180 ft) in diameter. The basic features for the mining of the internally supported cavities are shown in Figure 7-1. These features include:

1. 1 base level center crosscut drift. This drift, which is mined slightly off center, extends beyond the cavity a distance equal to approximately one radius. The drift provides personnel and equipment access to the cavity for mining of the raises, survey indexing, and muck removal.

2. 1 to 3 base level alcoves. These alcoves are mined directly under the raise and provide a temporary storage area for muck prior to removal along the access drift.

3. 1 to 3 raises. The raises, which are mined by conventional techniques, provide a means of center reference, personnel and equipment access, and muck removal.

The remaining mining detail pertains to the means of access to the cavity apex. For the internally supported caverns the two possibilities are shown in Figure 7-2. The simplest approach involves the mining of an exterior incline with a 15 to 20% grade. With this method, major equipment is driven up the incline once it is completed and mining of the apex is begun. However, when the mining of the apex is completed, major equipment can no longer be introduced into the cavity. Light equipment such as rockbolts, wire mesh, grout, and shotcrete can be lowered from the incline to the excavation level or can be introduced up the raises which also provides personnel access. This mining approach is satisfactory for the 24.4 and 36.6 m (80 and 120 ft) cavities since the excavation time for the smaller caverns is short and the probability of equipment

failure requiring more than minor repairs is very low. However, as the time required for cavity excavation increases with cavity size, the probability that a major equipment failure will occur, requiring the substitution of standby equipment, also increases. This problem is solved by the mining of a 15 to 20% internal spiral incline from the base level crosscut drift to the apex as shown in Figure 7-2. The spiral incline which disappears as the cavity excavation progresses downward, requires precise mining and somewhat complicates the excavation of the cavity. However, it allows personnel and equipment access to the construction level at any time during the excavation period, serves as a perimeter drift from which primary rock support could be placed prior to general excavation, and provides a means of obtaining detailed geological information of the cavity region prior to excavation. Although the spiral ramp provides advantages which are also desirable for the two smaller cavities, it becomes an unworkable approach due to the tight radii and limited available volumes characteristic of the smaller cavities.

Once access to the apex has been obtained by either approach, the internal cavities are assumed to be excavated by one to four Alpine miners depending upon the cavity size and stage of construction. The internal primary rockbolts are installed by Atlas Copco Simba H-221 hydraulic drill rigs which immediately follow the mechanical miners. Muck is removed by Wagner ST-3-1/2 or ST-5 scooptrams. Shotcrete is applied over the wire mesh using a rubber tire mounted Reed Guncrete machine.

7.2 EXTERNALLY SUPPORTED CAVITY CONSTRUCTION

An externally supported cavity design concept employing annular tendon galleries was considered for cavities 54.9, 73.2, and 91.4 m (180, 240, and 300 ft) in diameter. The basic features for the mining of the externally supported cavities are shown in Figure 7-3. These features include those listed below and those which are identical to features listed for the internally supported configurations are not described in detail.

1. 1 to 3 base level crosscut drifts.
2. 3 to 5 base level alcoves.
3. 3 to 5 raises.
4. 2 to 3 annular tendon galleries. These annular drifts provide a means of supporting the rock mass prior to excavation of the cavern, of monitoring extensometer and load cell readings throughout the excavation period, and of introducing additional rock support if necessary. It would also be feasible to reduce the number of annular tendon galleries and use the internal perimeter drifts (or the spiral ramps) as an access area for internally pre-supporting the rock.
5. A base level periphery drift. This drift provides a means of pre-anchoring the corner of the chamber in order to inhibit excessive rotation resulting from the excavation.
6. A crosscut drift from the upper annular drift. This drift provides a location from which to anchor the crown of the cavity.
7. Crosscut drifts from the lower two annular drifts. These crosscuts provide access to the cavity apex and to an intermediate excavation level during construction.

As with the larger internally supported cavity designs, access to the excavation level during construction is also an important factor for the externally supported cavity configurations. The crosscuts from the lower galleries provide single point access to the chamber similar to the exterior ramp approach to the 24.4 and 36.6 m (80 and 120 ft) diameter caverns. However, for these larger cavities, a partial internal spiral ramp (say from the base level to a level corresponding to 50 percent of the cavity height) is also included to increase chamber accessibility as the man-power and equipment demands also increase. Access to the tendon galleries is by inclined ramps as shown in Plate 6 which is typical of the externally supported cavities.

The cost and schedule estimates for the externally supported cavities are based upon using Alpine miners to excavate the caverns and using long hole drilling, muck removal and shotcrete equipment similar to that described for the internally supported cavities.

7.3 CONSISTENT BASIS COST AND SCHEDULE ESTIMATES

7.3.1 Cost

The estimated cost of construction for the various size hemispherical cavities evaluated in this report are given in Table 7-1 and include costs for geologic exploration, site access and development, cavity excavation, tendon purchase (for externally supported cavities only), and capital equipment. Costs for site access and development pertain to the mining of access drifts, base level drifts, raises and base level alcoves, annular tendon galleries, periphery drifts, spiral incline drifts, and any other mining or installation of tendons which must take place prior to the beginning of cavity excavation.

The development and excavation costs are determined from estimated manpower loadings throughout the excavation cycle and are based upon a unit manpower cost of $1400/man/week. This unit cost includes generally used materials such as rockbolts, wire mesh, and shotcrete. Since additional Alpine miners can sometimes be pulled into service during cavity excavation, as space and efficiency allows, with little or no increase in manpower loading, and since the mining of the core of the cavity can be less precise than that required near the surface of the chamber, the average cost to excavate a unit volume of rock decreases as the cavity size increases. This is reflected in the data shown in Figure 7-4 which was used to estimate cavity excavation costs. As a result, the variation in chamber excavation cost with span is nearly proportional to the square of the diameter, rather than to the cube of the diameter which would correspond to the volume of rock excavated.

The total cost figures are plotted versus cavity span in Figure 7-5 which reflect the consistent basis mining approach (Alpine miners only). It is apparent that some cost savings, on the order of 10 to 20 percent, could be

realized for the externally supported cavities if conventional drill and blast techniques were employed for excavation of the cavity core. Even so, it can be seen that the initial construction of an externally supported cavity configuration requires a greater dollar expenditure than does the construction of an internally supported design of the same size. However, stability and fracturing considerations suggest that at least the chamber roof should be supported externally for diameters greater than 45.7 m (150 ft) in order to avoid potentially costly problems resulting from the larger excavation and the greater potential of encountering intersecting joints and weakness planes. It is also recommended that support be installed from perimeter drifts or spiral ramps, prior to general excavation, in order to secure the lower portions of the chamber.

The estimated capital equipment requirements are shown in Table 7-2 and consider the additional equipment required to support the excavation. Where existing equipment is sufficient to support construction, no new equipment is specified. Some existing equipment such as grout pumps and short rockbolt jumbos are rail mounted and would require conversion to rubber tired carriage. Rail haulage and the use of rail mounted equipment, except at the base level, is not considered feasible since the length, and associated mining costs of access drifts with slopes acceptable for rail mounted equipment is prohibitive.

It should also be pointed out that current ventilation capabilities in the tunnel systems at NTS are probably not able to handle the increased air flows required of a project of this size. A new vent hole and fan system with a capacity on the order of 142 cubic meters per second (300,000 cfm) would cost an additional 3 to 3.5 million dollars.

7.3.2 Schedule

The consistent basis estimate of the time required to construct the various underground cavities are shown in Table 7-3. These time estimates reflect the working days required and include geologic exploration, site access and development, and cavity excavation. The time estimates consider a man loading sequence based upon the estimated mining, drilling and rock sup-

port installation rates shown in Table 7-4. It is noted that accuracy and alignment of the tendon drill holes is of extreme importance and the estimated tendon hole drilling rate shown in Table 7-4 reflects the additional care required.

The estimated working days for construction of the caverns is plotted versus cavity span in Figure 7-6 and shows the anticipated maximum and minimum time required for construction. The band width for each cavity size reflects the variation in schedule which could result from optimizing the excavation procedures and preparing a precise man-loading sequence for each cavity. It can be seen from Figure 7-6 that the variation of time with cavity span is somewhat more linear than the cost variation. This should be expected, since increased manpower can sometimes be efficiently employed to reduce the schedule with only a slight increase in total cost. The time estimates shown in Table 7-3 and Figure 7-6 are intended to reflect the increases in manpower for the larger cavity sizes and also the expected manpower fluctuation as the excavation of a cavity progresses to the lower stages. Since this is the case, it is felt that additional increases in manpower loading for the purpose of accelerating schedule would shortly result in increasing inefficiency of operation and sharply rising costs. It is expected that the final design effort for any anticipated cavity construction would include the optimization of the mining and excavation methods, capital equipment purchases, operation sequences, and the man-loading time history. Such a planning effort would generally result in the minimum construction time and nearly minimum costs.

7.4 OPTIMIZED INTERNALLY SUPPORTED CAVERN

In order to determine the magnitude of the impact of cavity planning and optimization on the estimated construction costs and schedule, an attempt was made to optimize the mining method and construction sequence for the 48.8 m (160 ft) cavern. Details of the development and excavation are shown in Plate 7 (F&S Drawing M3163-12) and are described below followed by a comparison of the optimized cost and schedule estimates to the consistent basis estimates. It should be noted that this is an optimization of the

excavation procedures only since the cavity detail (Plate 7) does not include the recommended annular tendon gallery.

All development, except parts of the main drift, are located inside the cavity perimeter. Conventional drill and blast methods and an Alpine miner are used in conjunction with Wagner ST-3-1/2 Load Haul Dump (LHD) units which transport and dump muck directly into rail cars or down raises to the rail level. The development sequence is as follows:

1. The main access drift is driven conventionally through the cavity area at the base level, along say the E-W cavity axis.

2. Two vertical raises for muck disposal, personnel access, and ventilation are driven conventionally to the 21.3 m (70 ft) and 18.3 m (60 ft) levels from the main drift at distances equal to 10.4 m (34 ft) and 14.5 m (47.5 ft) from the N-S cavity axis.

3. A perimeter drift and a 20% spiral ramp is excavated by Alpine to the 21.3 m (70 ft) level. Concurrently a LHD turn-around/dump alcove is excavated at the intersection of the ramp and the main drift.

4. Two short crosscuts are excavated by Alpine from the spiral ramp to the raises. These crosscuts provide LHD muck disposal points, ventilation, and emergency manway routes.

The excavation of the chamber similarly combines the use of Alpine miners and conventional techniques. The mining of the cavity apex from the 21.3 m (70 ft) level to the 27.6 m (90.6 ft) peak is by Alpine requiring muck ramps constructed by a Wagner ST-3-1/2 scooptram so that the Alpine can

reach the cavity back from the 21.3 m (70 ft) level. The cavity excavation below the 21.3 m (70 ft) level consists of mining a series of 3 m (10 ft) vertical disk shaped zones downward. The excavation sequence for the disk zones is as follows:

1. A circular doughnut-shaped level kerf is cut by Alpine miner along the cavity perimeter.

2. Concurrently with kerf mining, Atlas Copco Simba H-221 drill rigs which are located on the core bench level drill long 17 m (56 ft) holes in the crown of the cavern for fully grouted Dywidag rockbolts. Also concurrently short 3.7 m (12 ft) vertical blast holes are drilled in the bench on a 1.2 x 1.2 m (4 x 4 ft) circular pattern.

3. After completion of kerf mining and the installation of the rockbolts and wire mesh, a 7.6 cm (3 in) thickness of shotcrete is applied to the exposed cavity surface.

4. After completion of shotcreting, charges are placed in the vertical blast holes and the bench is removed with one shot. Muck is dozed or transported to the raises.

The four step mining cycle repeats itself for each 3 m (10 ft) high disk-shaped zone down to the floor level. After the cavity has been excavated to the base level, the Simba H-221 is used to drill 6.0 - 9.8 m (20 to 32 ft) holes in the cavity floor for fully grouted re-bar pins, completing the cavity support.

The cost and schedule estimate and the estimated capital equipment requirements to construct the 48.8 m (160 ft) cavity in this manner are given in Tables 7-5 and 7-6 respectively and are also noted in Figures 7-5 and 7-6.

Comparing the optimized and consistent basis cost estimates, the optimized cost of $8,109,000 for the 48.8 m (160 ft) cavity is approximately 14% less than the consistent basis value of $9,377,000. Based on these data, it is expected that the use of conventional mining methods for the other cavities over 48.8 m (160 ft) would result in a similar reduction in the cost estimate compared to the consistent basis estimate. As a result, a probable minimum cost line is also included in Figure 7-5. Similarly comparing the schedule estimates, the optimized schedule of 338 days is approximately 8% less than the consistent basis estimate of 360 days for the 48.8 (160 ft) cavern.

TABLE 7-1. ESTIMATED HEMISPHERICAL CAVITY COSTS ($1,000,000)

ITEM	CAVITY SIZE Meters (ft)						
	Internally Supported Cavities				Externally Supported Cavities		
	24.4(80)	36.6(120)	48.8(160)	54.9(180)	54.9(180)	73.2(240)	91.4(300)
Development Cost	.646	.782	2.075	2.287	3.750	8.397	10.514
Cavity Cost	.787	2.233	3.790	5.013	5.280	9.597	15.131
Tendon Purchase	—	—	—	—	2.065	5.094	9.421
Sub Totals	1.433	3.015	5.865	7.300	11.095	23.088	35.066
Capital Equipment	.415	.995	1.550	1.730	2.395	2.965	2.991
Sub Totals	1.848	4.010	7.415	9.030	13.490	26.053	38.057
Contingency	10% .185	10% .401	15% 1.112	15% 1.355	15% 2.023	20% 5.210	20% 7.611
Exploration	.850	.850	.850	.850	.850	.850	.850
Totals	2.883	5.261	9.377	11.235	16.363	32.113	46.519

TABLE 7-2. ESTIMATED NEW CAPITAL EQUIPMENT REQUIREMENTS
NUMBER AND (K $)

Item	Cavity Size						
	80'	120'	160'	180' Int	180' Ext	240'	300'
Tendon Stressing Rams	-	-	-	-	6 (37)	6 (37)	6 (37)
Grout Pumps	2 (15)	2 (15)	2 (15)	2 (15)	2 (15)	2 (15)	2 (15)
L H D	1 (70)	2 (140)	3 (210)	4 (280)	4 (280)	5 (350)	5 (350)
Rockbolt Jumbos	1 (120)	1 (120)	1 (120)	1 (120)	1 (120)	1 (120)	1 (120)
Tendon Jumbos	-	-	-	-	4 (600)	4 (600)	4 (600)
Shotcrete Machines	1 (100)	1 (100)	1 (100)	1 (100)	1 (100)	1 (100)	1 (100)
Alpine Miners	0	1 (400)	2 (800)	2 (800)	2 (800)	3 (1,200)	3 (1,200)
Support Trucks	1 (85)	1 (85)	1 (85)	2 (170)	2 (170)	2 (170)	2 (170)
Personnel Carriers	0	1 (60)	2 (120)	2 (120)	2 (120)	2 (120)	2 (120)
Auxiliary Vent Equip.	(0)	(25)	(50)	(50)	(50)	(100)	(100)
Extensometers	(25)	(50)	(50)	(75)	(75)	(100)	(100)
Load Cells	-	-	-	-	16 (28)	30 (53)	45 (79)
TOTALS K $	$415	$995	$1,550	$1,730	$2,395	$2,965	$2,991

TABLE 7-3. ESTIMATED HEMISPHERICAL CAVITY SCHEDULE (Working Days)

ITEM	Internally Supported Cavities				Externally Supported Cavities		
	24.4(80)	36.6(120)	48.8(160)	54.9(180)	54.9(180)	73.2(240)	91.4(300)
Development and Tendon Installation	35	43	99	102	122	231	268
Cavity Excavation	40	100	150	188	199	286	400
Sub Total	75	143	249	290	321	519	668
Contingency 10%	8	14	25	29	32	52	67
Exploration	86	86	86	86	86	86	86
Total	169	243	360	405	439	655	821

TABLE 7-4. ESTIMATED MINING, DRILLING AND INSTALLATION RATES

Drift Mining/Machine (Alpine Miner)

- $13^w \times 11^h$ (drifts) = 10'/shift (level to 15% grade).
- $13^w \times 13^h$ (incline) = 2 passes (13 x 11 [10'/shift + 13 x 2 @ 17'/shift).
- $16^w \times 14^h$ (donut) = 3 passes (11 x 11 @ 10'/shift + 5 x 11 @ 15'/shift + 16 x 3 @ 17'/shift).
- $10^w \times 8^h$ (cavity) = 17'/shift.
- 9 x 6 (Raises) = 4'/shift (Drill & Blast).

Internal Rockbolts

- Rockbolt Drilling (60 ft) = 10/shift/machine
- Rockbolt Drilling (48 ft) = 15/shift/machine
- Rockbolt Drilling (36 ft) = 20/shift/machine
- Rockbolt Installation = 2 x drilling rate
- Rockbolt Grouting = 33/shift

Tendons

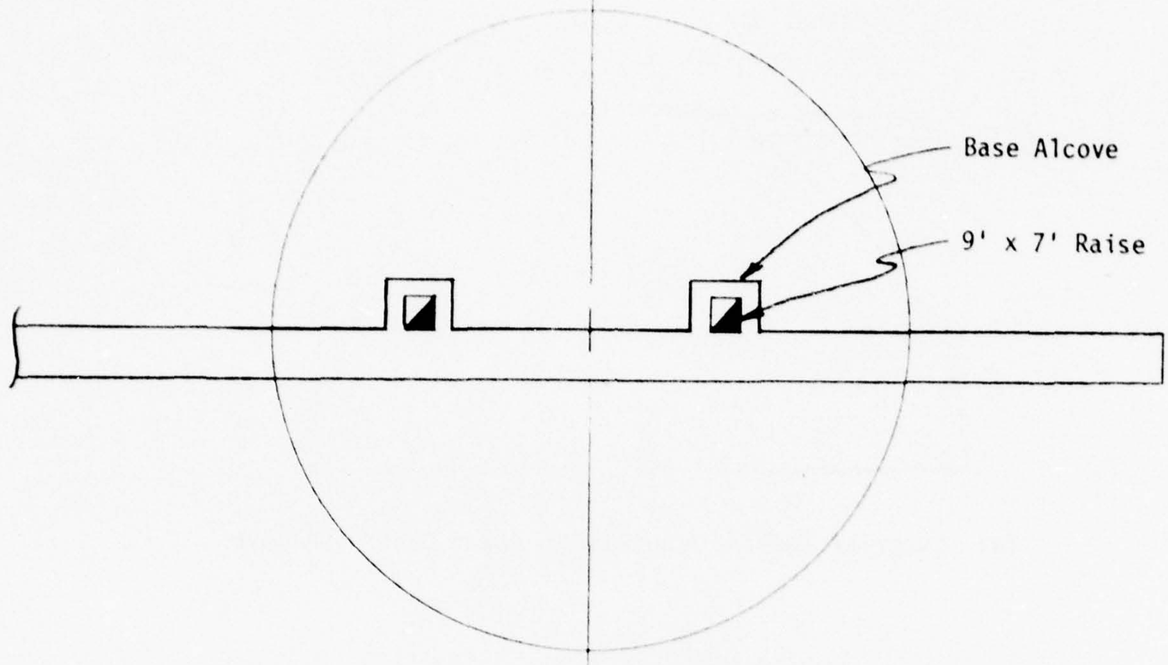
- Tendon Drilling (100 ft) = 2/shift/machine (including survey and setup time)
- Tendon Installation = 20/shift

TABLE 7-5. OPTIMIZED COST AND TIME ESTIMATE - 48.8 m (160 ft) CAVITY

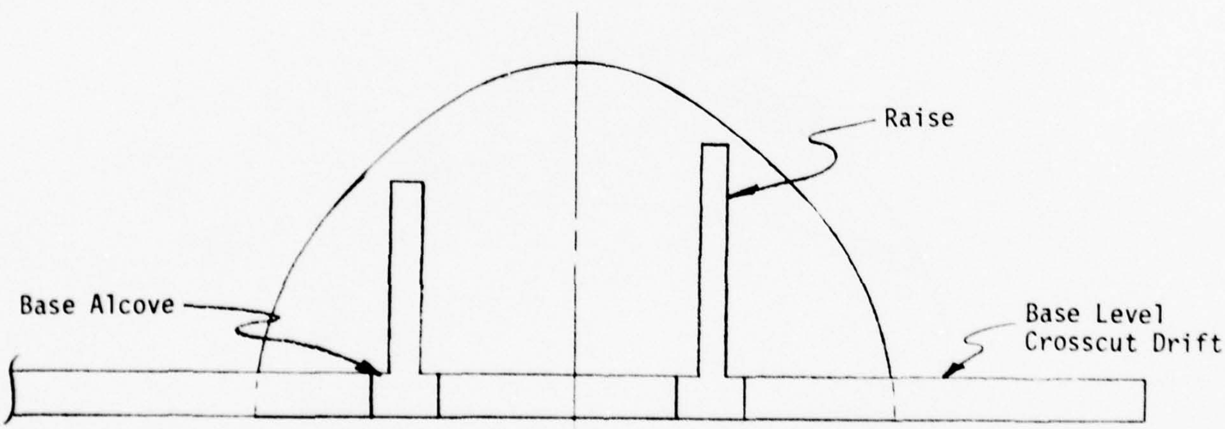
	Costs($1,000,000)	Time (days)
Development (75 men/day crew)	.989	47
Cavity Excavation & Support (90 men/day crew)	4.576	182
Sub Total	5.565	229
Capital Equipment	1.034	
Sub Total	6.599	
Contingencies (10%)	.660	23
Exploration	.850	86
Total	8.109	338

TABLE 7-6. ESTIMATED CAPITAL EQUIPMENT REQUIREMENTS - 48.8 m (160 ft) Optimized Cavity

Item	No. Req'd.	Unit Cost ($K)	Total Cost ($K)
Wagner ST-3½ Scooptram	2	$ 85.0	$170.0
Wagner UT-45A Utility Truck	2	35.0	70.0
Cat D5B Bulldozer	1	50.0	50.0
Atlas Copco Simba H221 Hydraulic Drill Rig	2	145.0	290.0
Spare Drill Consumable Parts	1	24.0	24.0
			15.0
Reed Guncrete Shotcrete Machine	1	15.0	15.0
		TOTAL	$634.0

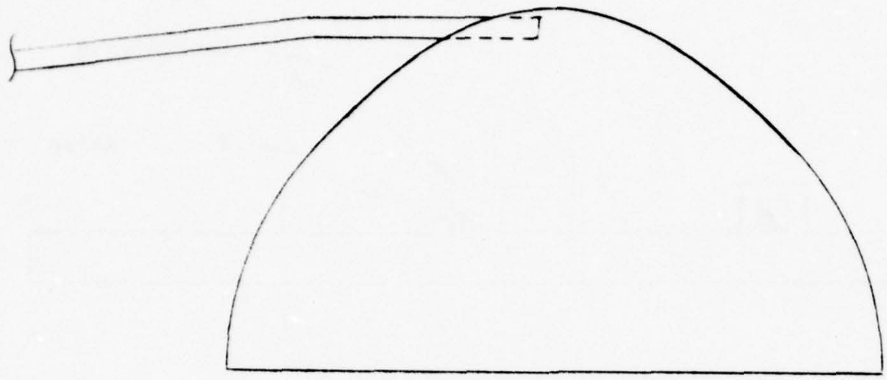


PLAN VIEW

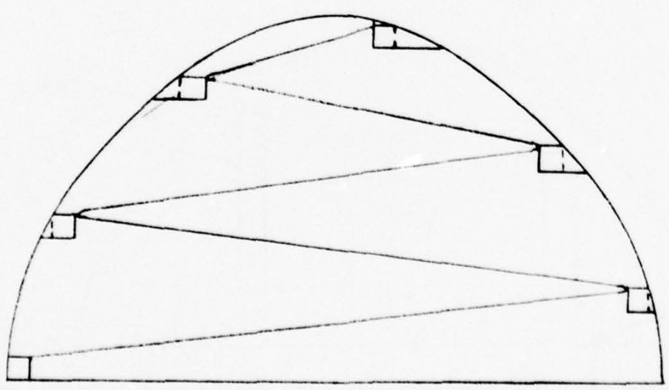


ELEVATION VIEW

FIGURE 7-1. BASIC FEATURES FOR CONSTRUCTION OF INTERNALLY SUPPORTED CAVITIES



(a) Exterior Incline Approach to Apex Raise & Alcove



(b) Interior Spiral Drift Approach to Apex Periphery Drift

FIGURE 7-2. APEX APPROACH FOR INTERNAL SUPPORT CAVITIES

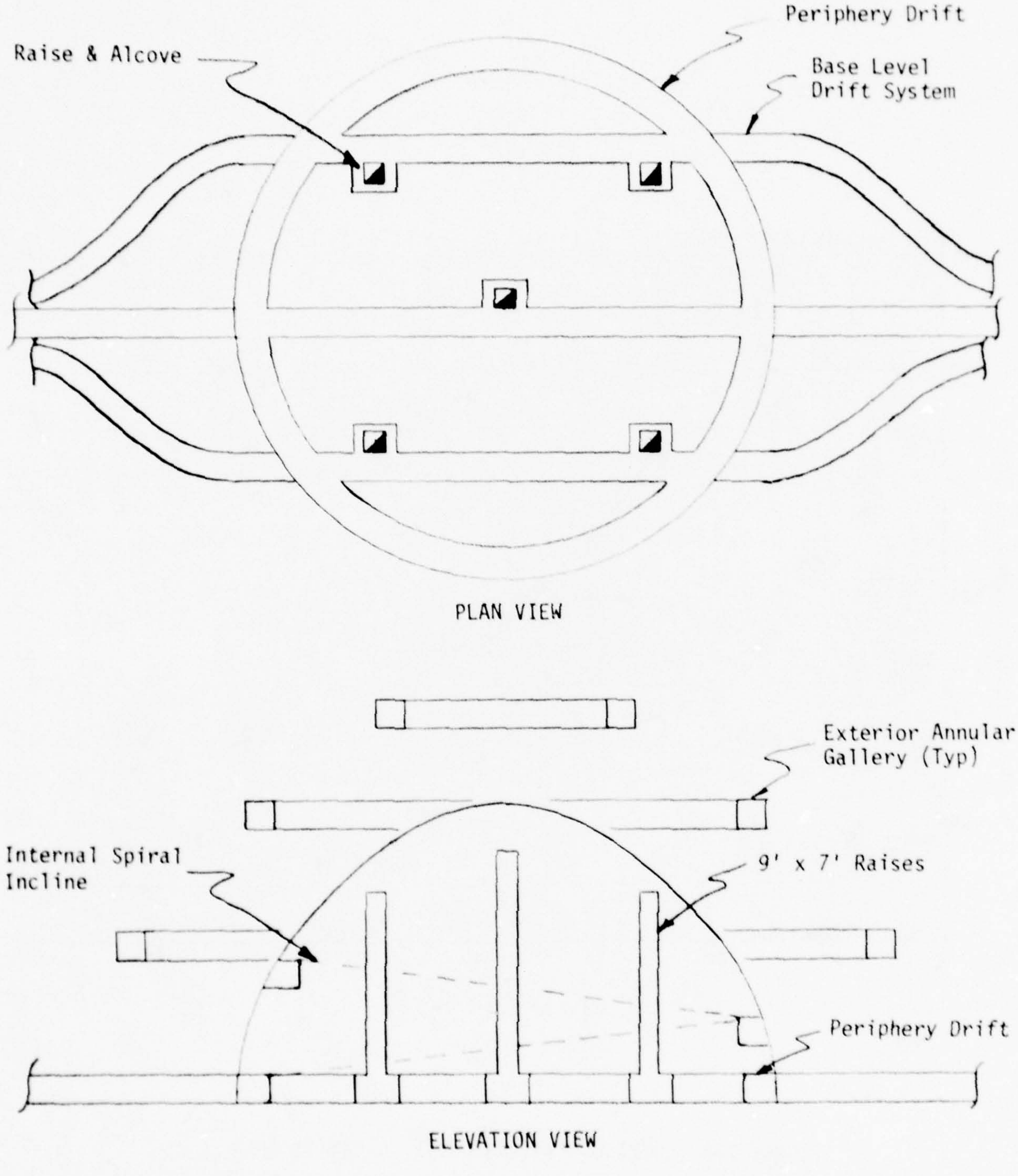


FIGURE 7-3. BASIC FEATURES FOR CONSTRUCTION OF EXTERNALLY SUPPORTED CAVITIES

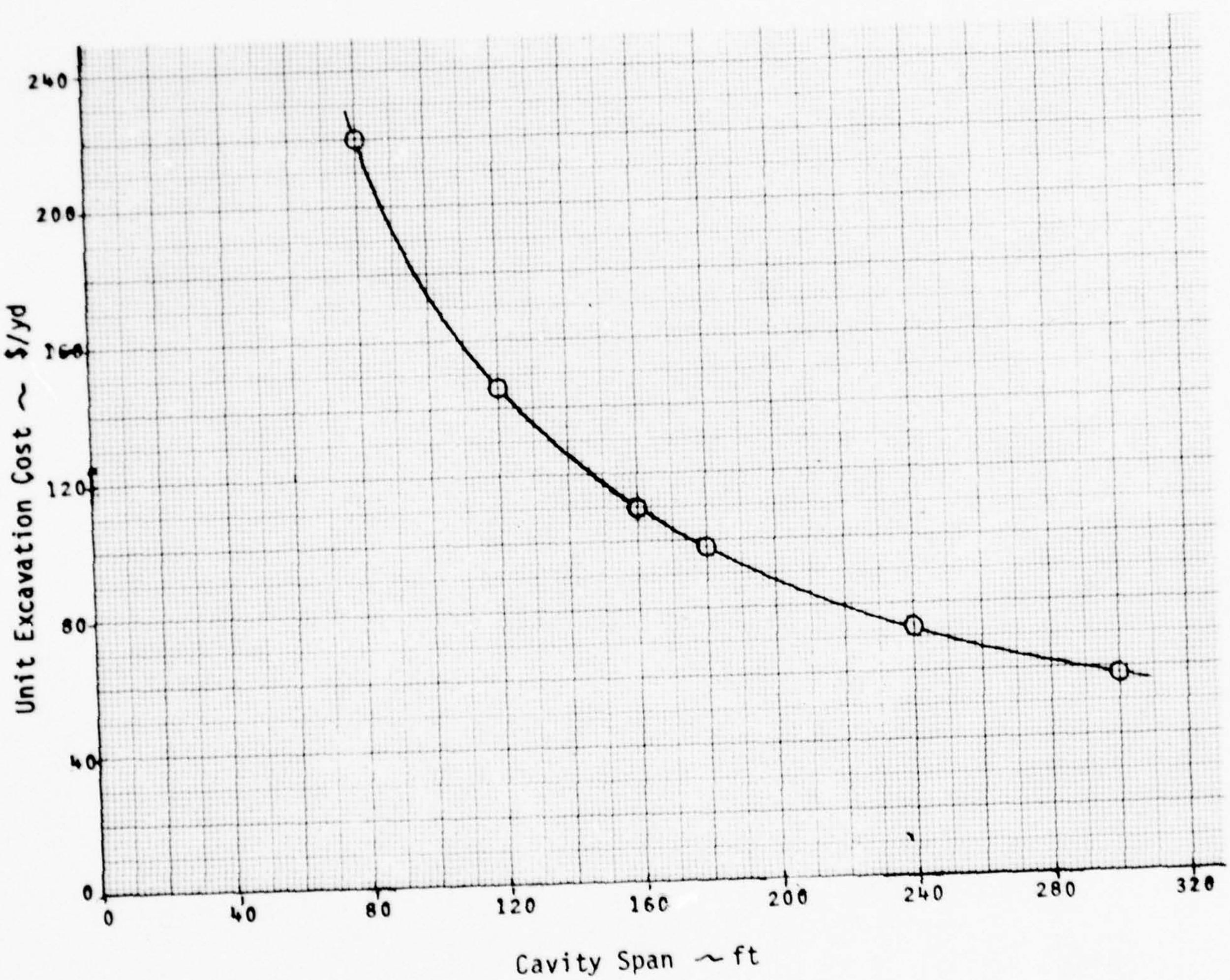


FIGURE 7-4. UNIT EXCAVATION COSTS VS CAVITY SPAN

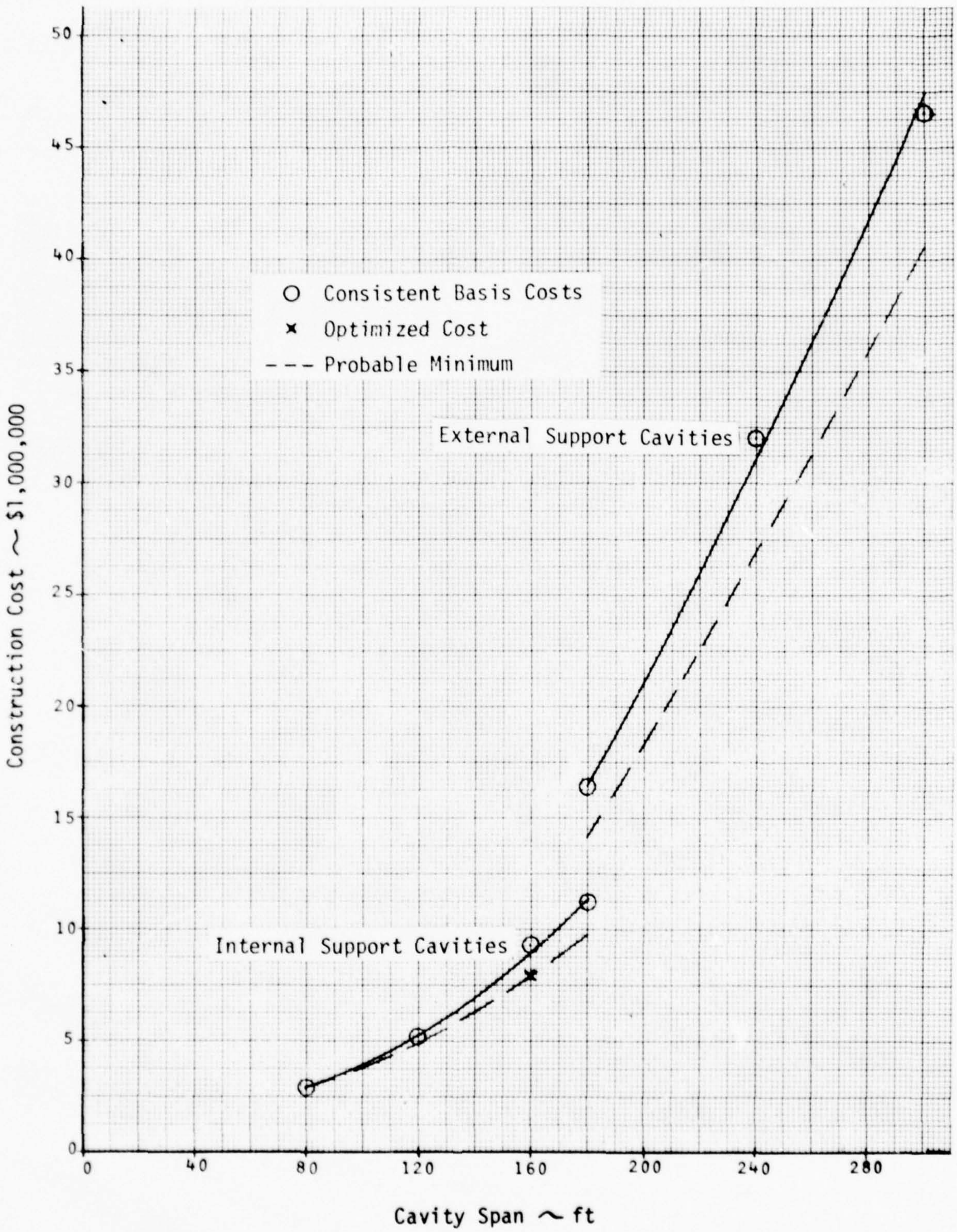


FIGURE 7-5. CONSTRUCTION COSTS VS CAVITY SPAN

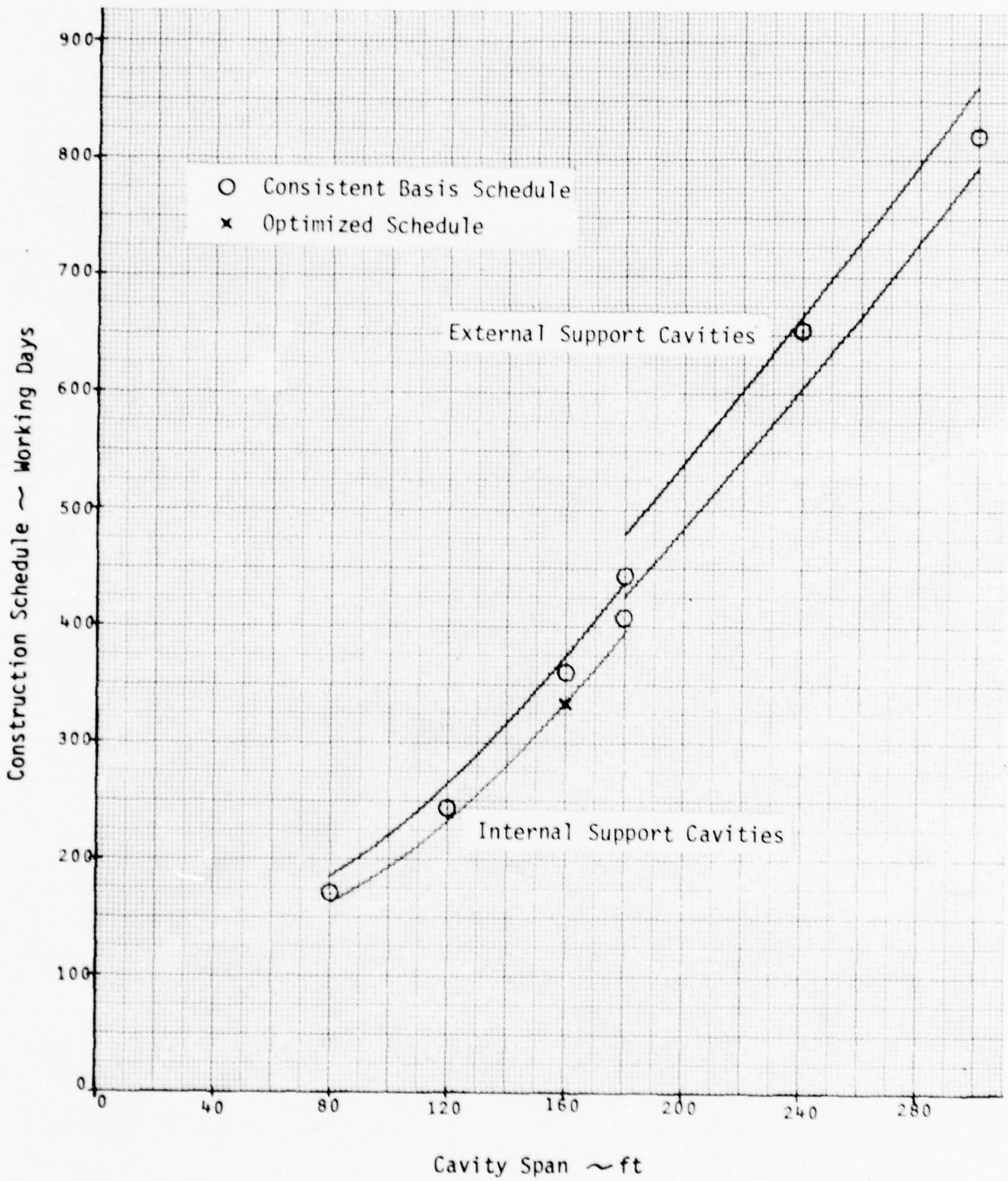


FIGURE 7-6. CONSTRUCTION WORKING DAYS VS CAVITY SPAN

8. CONCLUSIONS

The cost and feasibility evaluation program documented in this report has been concerned with identifying and assessing the effect of important parameters pertaining to the design and construction of large underground cavities under Rainier Mesa at the Department of Energy's Nevada Test Site. Several of the more important parameters are listed below and the findings of this study pertaining to each are summarized in the subsections which follow.

- State-of-the-Art in Underground Cavity Excavation
- Geological Character of Rainier Mesa
- Lithology of Tunnel Beds 4
- Cavity Shape and Support System Design
- Estimates of Cavity Excavation Cost and Schedule

A number of conclusions and recommendations are included in each subsection. Following these, overall conclusions are discussed concerning the maximum practical chamber size considering schedule, dollar, and state-of-the-art constraints

8.1 STATE-OF-THE-ART IN UNDERGROUND CAVITY EXCAVATION

As discussed in Section 3 of this report most of the past experience with the excavation of large underground chambers is associated with hydroelectric power stations. Spans for the machine halls of these power stations range up to 33.5 m (110 ft) in width. The machine halls are characteristically high and long compared to their span as indicated by the Portage Mountain Powerhouse (British Columbia) which is 46.6 m (153 ft) high by 20.4 m (67 ft) wide by 271.3 m (890 ft) long and the Waldeck II machine hall which is 33.5 m (110 ft) wide by 53.9 m (177 ft) high by 103.6 m (340 ft) long. Several of these facilities have been constructed in bedded rock formations similar to that found in Rainier Mesa although the intact rock strength has generally been significantly higher. The hemispherically shaped Red Hot and Deep Well cavities previously constructed in the G-tunnel complex in Rainier Mesa were excavated

in tuffs similar to those expected at the proposed cavern site and were 36.6 m (120 ft) in diameter. It is therefore concluded that a hemispherically shaped cavern on the order of 45.7 to 54.9 m (150 to 180 ft) in diameter would correspond to the level of current practice, being generally equivalent to the largest of the machine halls constructed in bedded deposits and somewhat larger than the Red Hot-Deep Well cavities.

The rock support system and the construction techniques for larger cavities between 54.9 and 91.4 m (180 and 300 ft) in diameter are very similar to that which has been successfully used in the past on powerhouse facilities and therefore must be considered feasible and within the state-of-the-art being simply a scaled up version. Annular tendon galleries and internal drifts are recommended in all cases as a means of providing access for the placement of rock support and instrumentation prior to general excavation of the cavern. The pre-support provided by this system will limit ground movements and loosening along stress-induced fractures as well as natural joints and bedding planes. The external galleries or internal drifts may be sited in the vicinity of major known geologic rock defects in order to provide a means of positive pre-support of adverse features such as faults and friable bedding planes. In summary, pre-support and instrumentation installed from the galleries and drifts will provide a degree of control not experienced in the Red Hot and Deep Well caverns constructed 12 years ago. Thus, the larger caverns can be constructed in a more controlled manner than the previous caverns excavated in tuff.

Potential unknown size effects and other construction technique problems such as the accuracy of long-hole drilling limit the confidence level pertaining to the successful construction of these larger cavities without some further testing. Therefore, it is concluded that the practical state-of-the-art is definitely in the 45.7 to 54.9 m (150 to 180 ft) diameter range and may extend to a diameter of 64.0 m (210 ft) while the 73.2 and 91.4 m (240 and 300 ft) diameter caverns are felt to be moderate and significant extensions of the practical state-of-the-art respectively. However, it should also be noted that some reasonably simple construction tests might increase the confidence level for the successful excavation of these larger cavities.

8.2 GEOLOGICAL CHARACTER OF RAINIER MESA

The Rainer Mesa tuff which is located at the tunnel level of the various tunnel systems has generally been found to be of good quality being highly homogenous and exhibiting a high Rock Quality Designation (RQD) index. Furthermore, the tuff also exhibits a low core index indicating the absence of a large number of joints or fault zones.

North-south striking near vertical fault zones, consisting of vertical joints and fractures in a 0.6 to 6.0 m (2 to 20 ft) wide zone should be expected on a horizontal spacing of approximately 60 to 120 m (200 to 400 ft). It may be possible to avoid major zones in the vicinity of the cavern; however, it has been assumed that at least one vertical fault zone will intersect the cavern or will be located behind the cavern wall. Individual fault planes or joints may be located between the major fault zones. Although the faults will be continuous over distances in excess of 15 m (50 ft), the planes of weakness in the fault zone will tend to be wavy and irregular, giving some interlocking to the rock mass.

The ratio of the strength of the tuff to the in-situ stresses is low enough that stress-induced fractures will form to a depth of 5 to 10 ft behind the walls and temporary floor of the cavern as excavation is carried out. Loosening of the slabs formed by these fractures can be minimized by prompt support or pre-support of the ground. New fractures, combined with natural joints and bedding plane weaknesses can also form unstable wedges of rock, unless properly supported.

8.3 LITHOLOGY OF TUNNEL BEDS 4

It was stated in the problem description that the cavity was to be located in tunnel beds 4 of Rainier Mesa. Based upon stability and rock support considerations, the cavern should be sited in tuffs which have few friable zones and which are free of clayey zones subject to swelling. Massive tuffs are preferred to closely bedded tuffs particularly for the chamber crown and floor levels. A review of the lithologic logs indicates that tunnel beds 4G through 4K are reasonably free of clayey zones except in the upper portions of

tunnel bed 4K where weak friable zones should be expected at a vertical spacing of approximately 15 m (50 ft). These zones are typically 1.5 m (5 ft) thick and at least three (3) friable zones should be expected in the cavern interval of a 91.4 m (300 ft) diameter cavity. Closely spaced bedding plane partings should be expected at approximately 3 to 12 m (10 to 40 ft) vertical intervals. By proper siting of the cavern it should be possible to avoid significant swelling zones in the tuff.

Tunnel bed 4G is identified as a massive layer satisfactory for location of the chamber floor level. Conversely the upper level of unit 4K, identified above as having broad clayey zones, should be avoided if possible.

8.4 CAVITY SHAPE AND SUPPORT SYSTEM DESIGN

As a result of the need to maximize the minimum radius to the surface of the cavity, a hemisphere was chosen as the starting point for the shape of the cavity. However, due to stability and rock support considerations, it was considered advantageous to provide a smaller radius at the cavity apex. Therefore, a final shape was selected which consisted of an intersecting paraboloid and hemisphere as shown in Figure 5-1. The selected configuration has a planar horizontal floor level promoting ease of construction and functional use. The selection of the final shape and cavity orientation is based to a great degree on the experience gained during the excavation of the Red Hot/Deep Well cavities as are many of the details of the rock support system. However, additional analyses which were outlined in Section 6 of this report should be a part of the final design effort.

The design of the rock support system for the large cavities is based on previously used conventional techniques. This is particularly true of the internally supported cavities. The depth of rock surrounding the cavern which is influenced by the presence of the opening is conservatively judged to be equal to 20 percent of the span. As a result the internally installed rockbolt length is specified to be at least 30 percent of the cavity span at the crown. The supports will be installed on a square pattern with a 1.5 to 1.8 m (5 to 6 ft) spacing between rockbolts followed by application of wire mesh and a 7.6 cm (3 in) coating of shotcrete.

Similarly, the externally supported designs employ conventional techniques. However, unlike the internal support designs, past experience does not extend to the cavern sizes evaluated in this report. The tendon galleries, which provide a means of pre-supporting the rock mass above the cavern prior to general excavation, are located at a minimum distance of twice the rock load depth from the cavity perimeter and the tendons extend to within approximately .6 m (2 ft) of the surface. As a result, short personnel rockbolts are required to support the cavity surface and to provide a means of attaching the wire mesh prior to application of the shotcrete. Both the externally installed tendons and the internally installed personnel rockbolts should form a square pattern with a 1.5 m (5 ft) spacing. The spacing of the externally installed tendons may vary, depending on the tendon capacity and the configuration of the annular galleries. Tendon spacing should not exceed 6.1 m (20 ft) at the cavern walls.

The primary rock support, consisting of long tendons and rockbolts is designed to hold the major wedges of rock in place that are formed by combinations of bedding plane weaknesses, joints and or faults, and new fractures. Primary support installed from initial drifts and galleries is intended to limit deep loosening behind the walls, including below the floor. The secondary support at the cavern wall, consisting of short rockbolts and shotcrete, is designed to hold the skin of the cavern in place, provide for personnel safety, and limit loosening and drying of the tuff.

Pre-supporting of the rock mass is essential for the larger cavity sizes in order to prevent rock fracturing and instability which can occur as a result of excavation before rock support can be installed from within the cavity. It is recommended that cavities larger than 45.7 m (150 ft) in diameter include at least one tendon gallery to pre-support the crown. Portions of the sidewalls of the smaller cavities may be supported by means of internally installed rockbolts.

The various components of the proposed support system should be tested prior to final selection. The testing of tendons, drilling of tendon holes, and development of bearing pads would be part of this program. A program of testing

and design review prior to construction is recommended, in order to establish the best methods for supporting, monitoring and excavating caverns of this size.

8.5 ESTIMATES OF CAVITY EXCAVATION COST AND SCHEDULE

Plots of the variation of construction cost and schedule (working days) versus cavity span are shown in Figures 7-5 and 7-6 respectively. These estimates are based upon the Alpine miner excavation of caverns which are either completely internally or completely externally supported. Costs related to the externally supported designs rise quickly in relation to the internally supported designs reflecting the increased development mining required and cost of the tendons. A probable minimum cost variation is included which assumes the use of conventional drill and blast techniques for excavation of the cavity core as opposed to excavation of the entire cavity by means of Alpine miners. Similarly the construction schedule indicates the potential variation which can be expected as a result of optimizing the excavation plan. The schedule is presented in terms of working days based upon an average man-loading. Elapsed calendar time will vary based upon contractor priorities, extended work weeks and other factors. Since certain tasks can be accomplished simultaneously or by increased man-power loading for the larger cavity sizes, the schedule variation does not exhibit the sharp increase between internal and external support designs seen for construction cost.

8.6 OVERALL RECOMMENDATIONS

Based on the data presented in this report, the recommendations concerning the maximum practical size of a cavity excavation in Rainier Mesa must be mostly dependent upon the needs of the experimenters. It is judged that the excavation of even a 91.4 m (300 ft) cavern is within the technical state-of-the-art even though the confidence level might be somewhat low without some additional testing and analysis. Once the minimum size has been established based upon the experiment needs, dollar and schedule constraints then are expected to govern. If the minimum required size is less than 54.9 m (180 ft), the successful construction confidence level will be very high, the cost less than 12 million dollars, and the schedule on the order of 400 days or less based upon the use of a single tendon gallery for cavities between 45.7 and 54.9 m (150 and 180 ft) in diameter. For larger minimum requirements, the

costs and schedule rise rapidly as well as uncertainties related to unknown factors. Therefore, it is recommended that the maximum diameter considered for any construction in the near future be limited to 54.9 m (180 ft) which is a significant extension of current practice at NTS being 3.3 times the excavated volume of the Red Hot and Deep Well cavities and which provides a high degree of confidence for successful construction.

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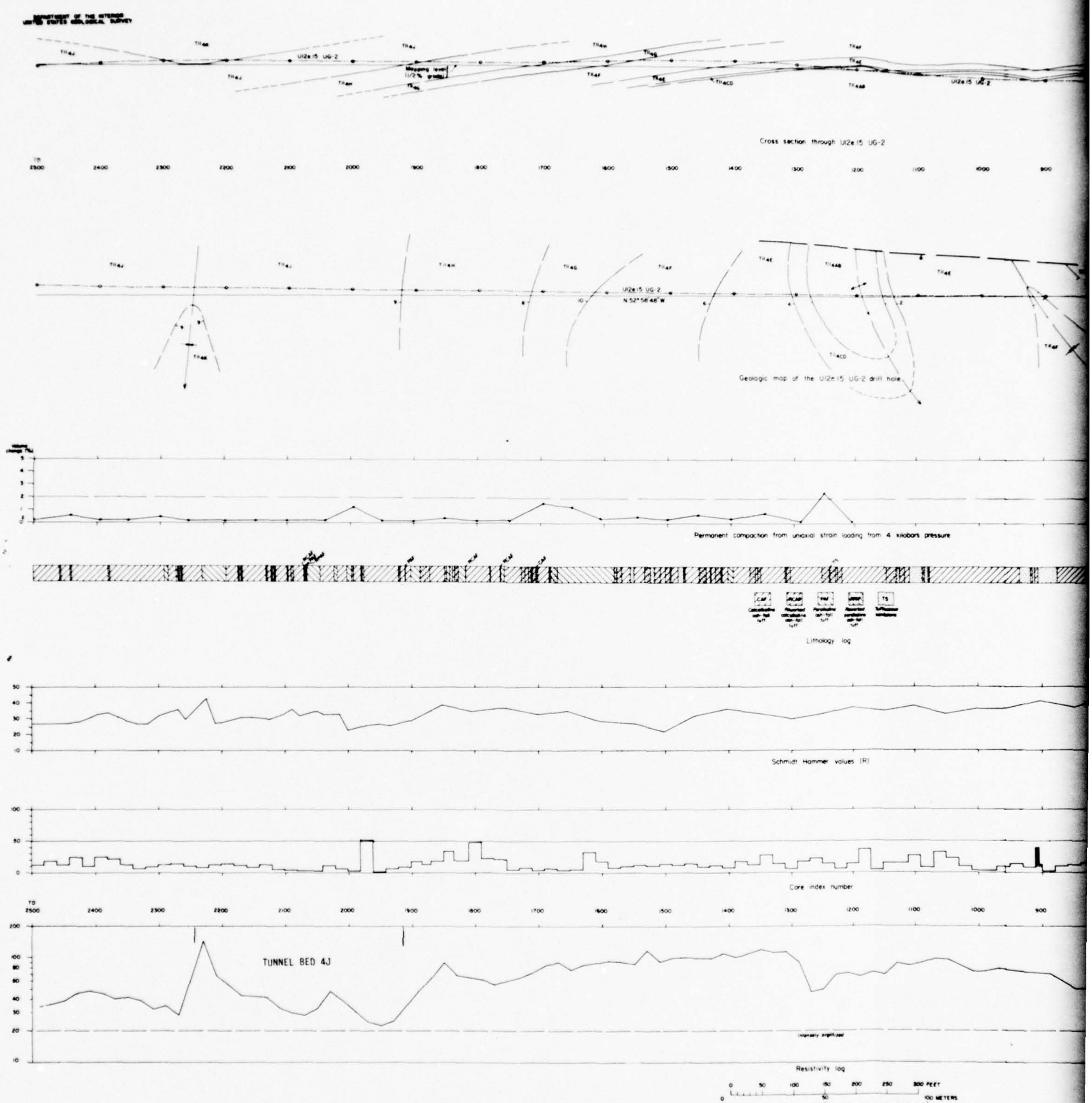

PLATE 1. GEOLOGY AND GEOPHYSICS OF THE U12e.15 UG-2
HORIZONTAL DRILL HOLE.

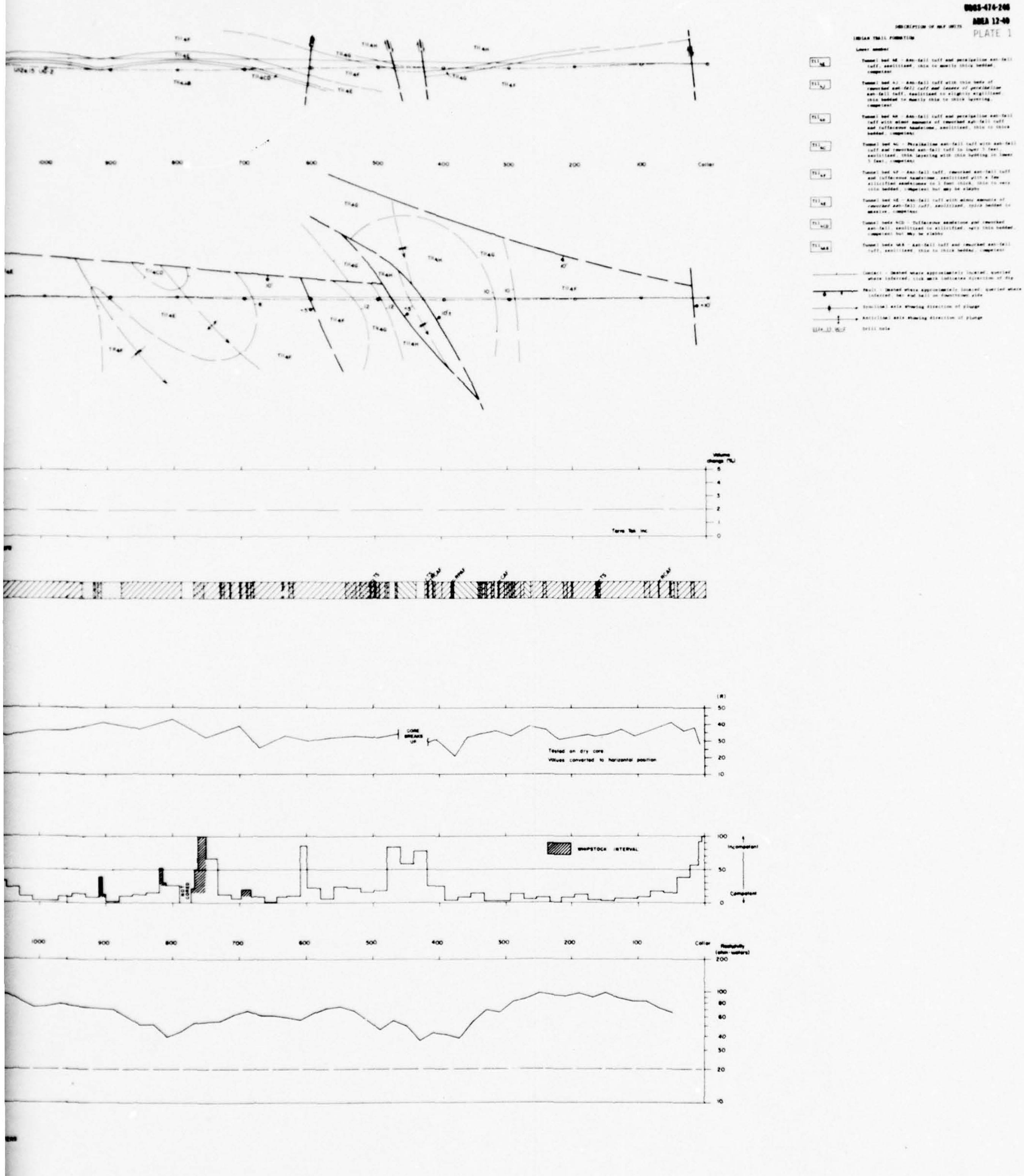
PLATE 1
DESCRIPTION OF MAP UNITS
INDIAN TRAIL FORMATION
Lower member
Drill hole
Collar
Volume change (%)
Terra Tek Inc
CORE BREAKS UP
Tested on dry core
Values converted to horizontal position
WHIPSTOCK INTERVAL
Incompetent
Competent
Resistivity (ohm-meters)

2

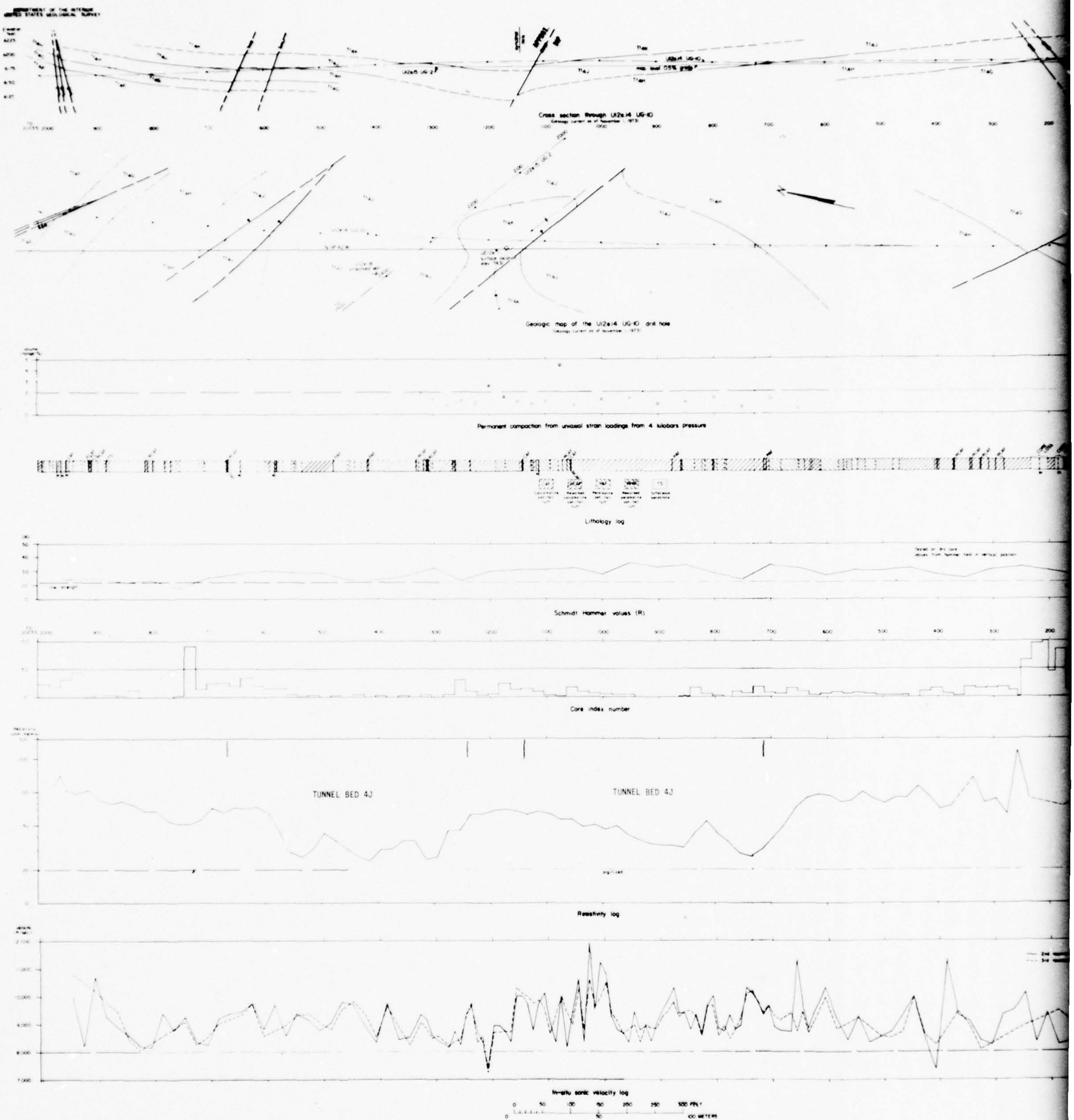

PLATE 2. GEOLOGY AND GEOPHYSICS OF THE U12e.14 UG-10 HORIZONTAL DRILL HOLE.

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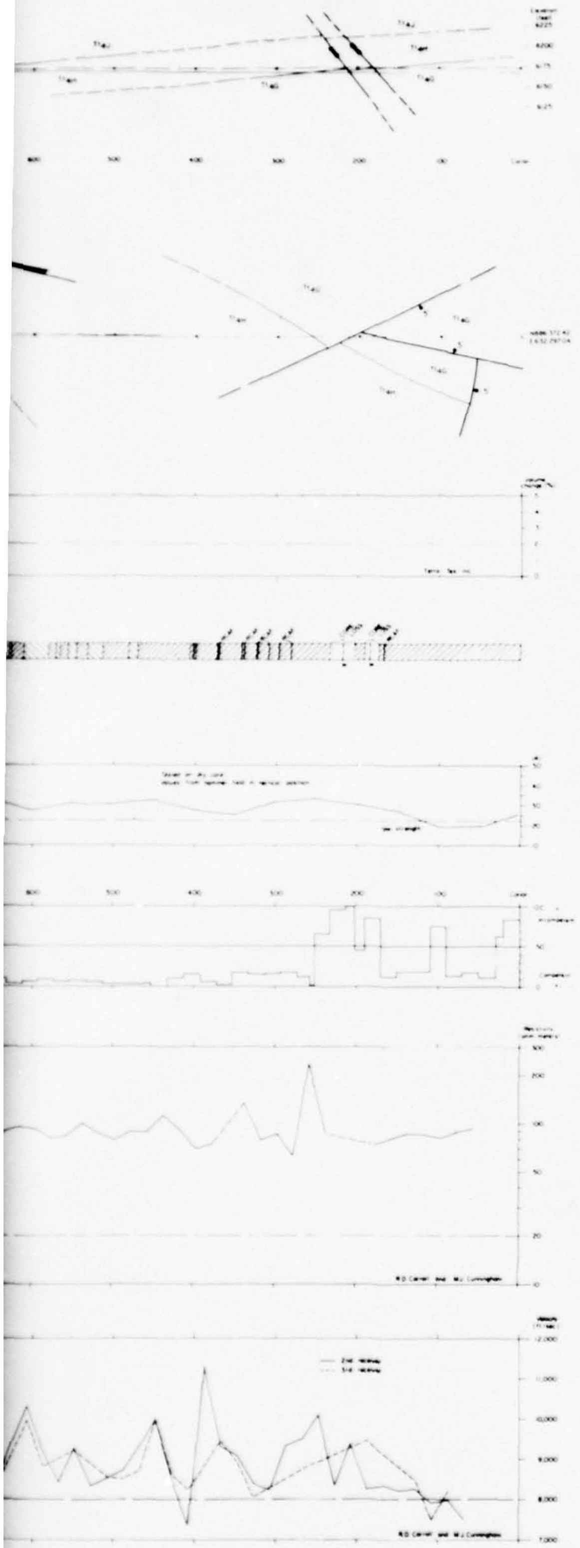
USGS-474-246
AREA 12-49
PLATE 2



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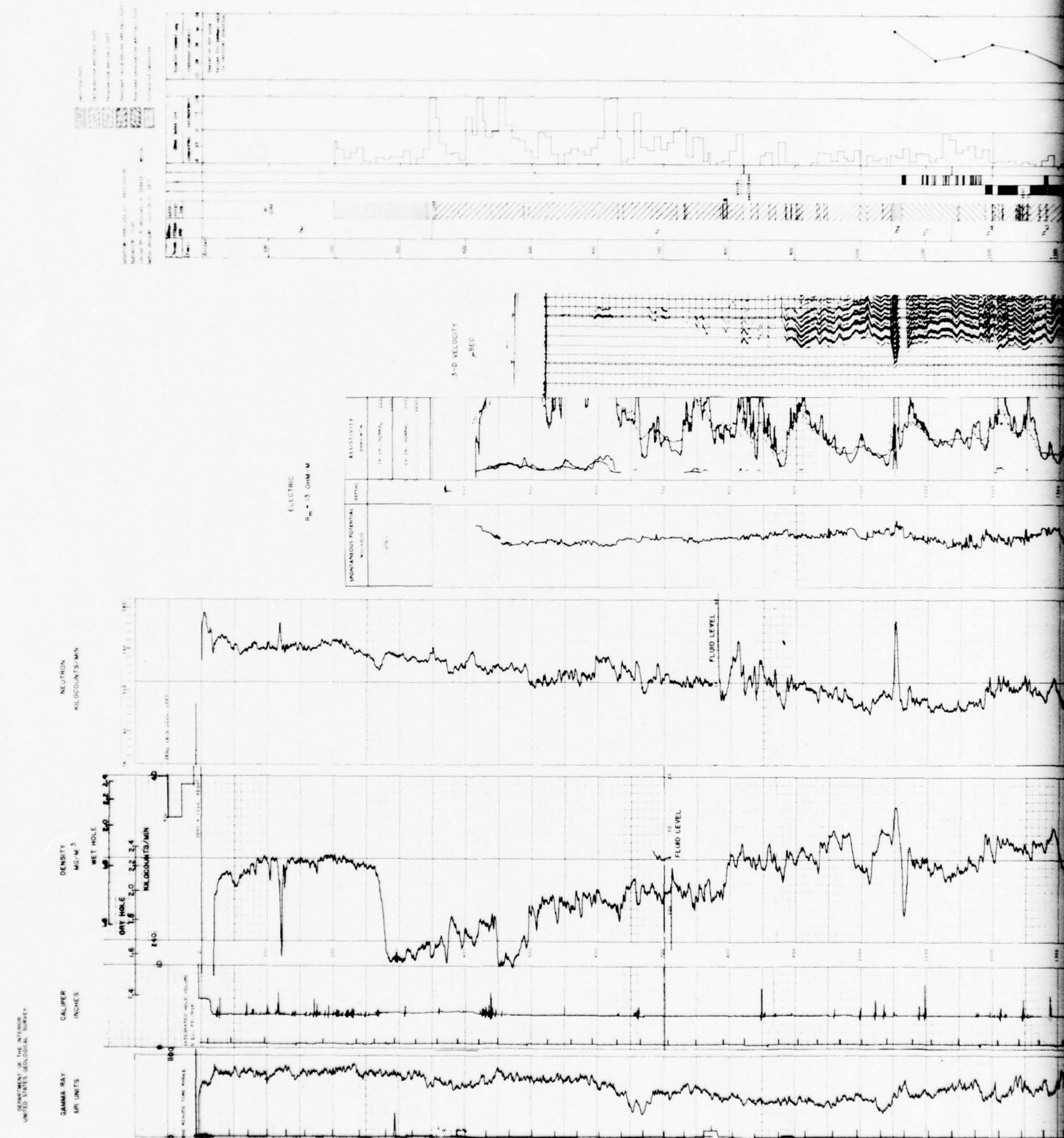

DEPARTMENT OF THE INTERIOR
UNITED STATES GEOLOGICAL SURVEY
GAMMA RAY
API UNITS
CALIPER
INCHES
DENSITY
MG/M3
WET HOLE
DRY HOLE
KILOCOUNTS/MIN
FLUID LEVEL
NEUTRON
KILOCOUNTS/MIN
FLUID LEVEL
ELECTRIC
SPONTANEOUS POTENTIAL
RESISTIVITY
3-D VELOCITY
μSEC

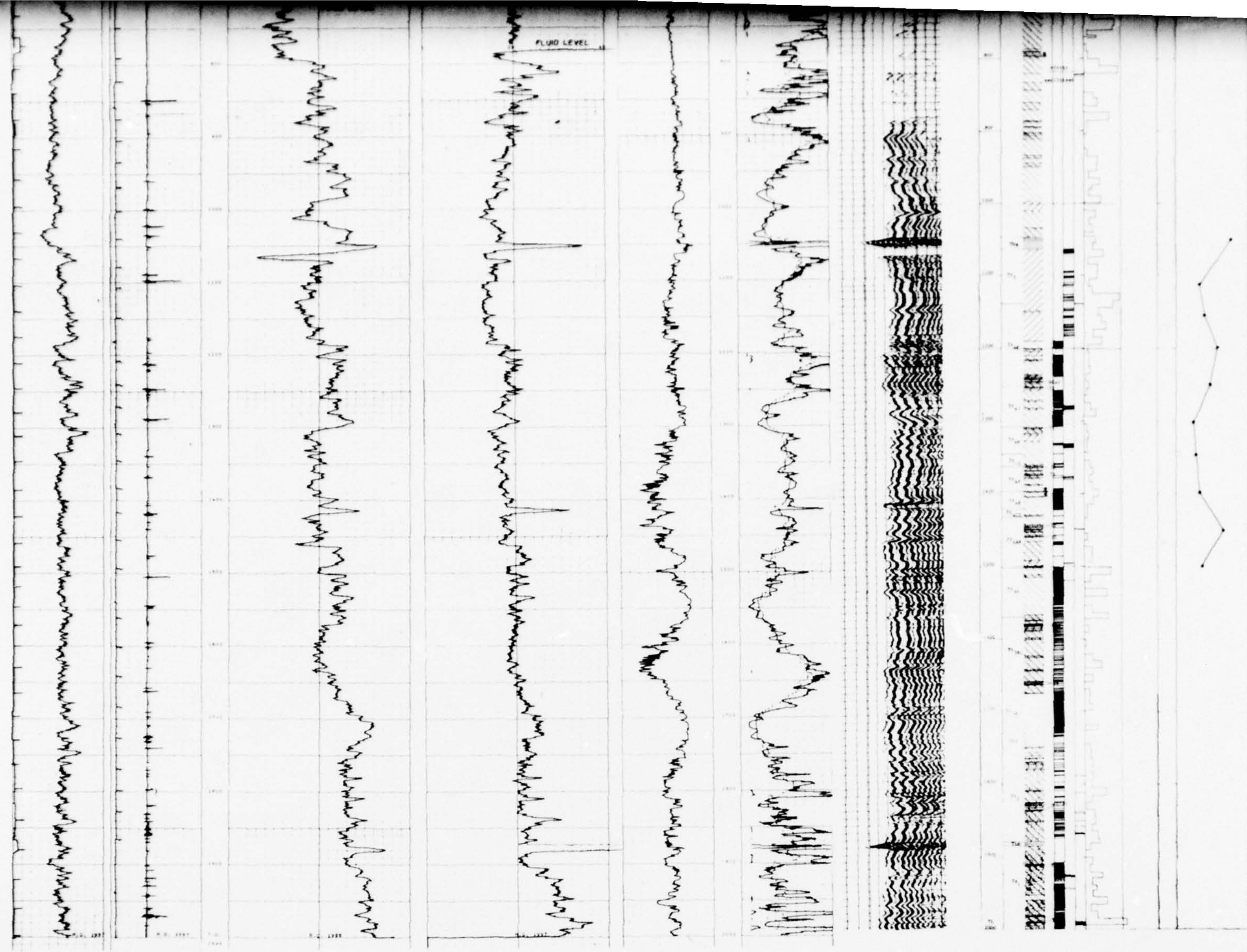
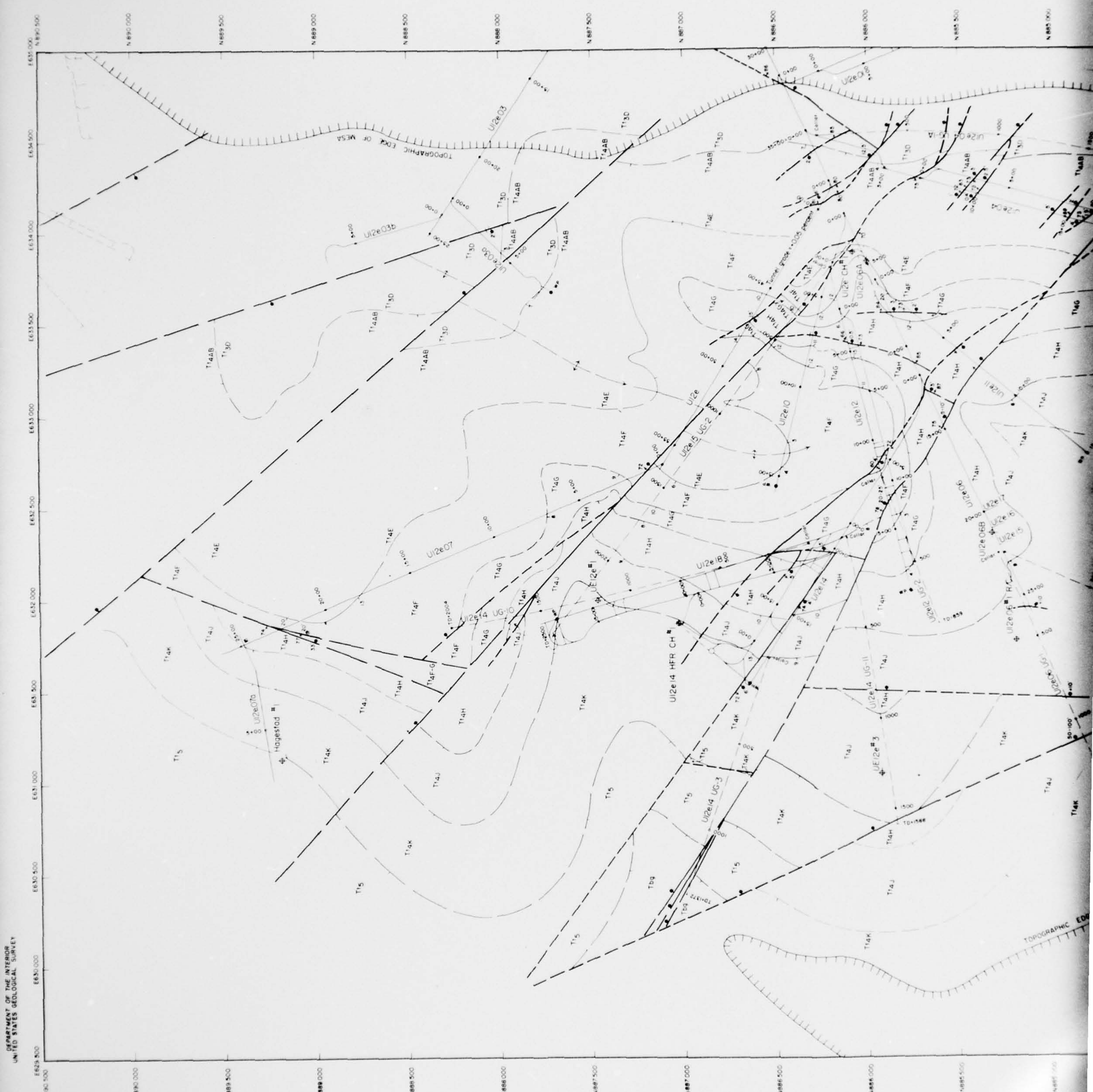


PLATE 3. GEOLOGY AND GEOPHYSICS OF THE U12e#1 VERTICAL DRILL HOLE.

2



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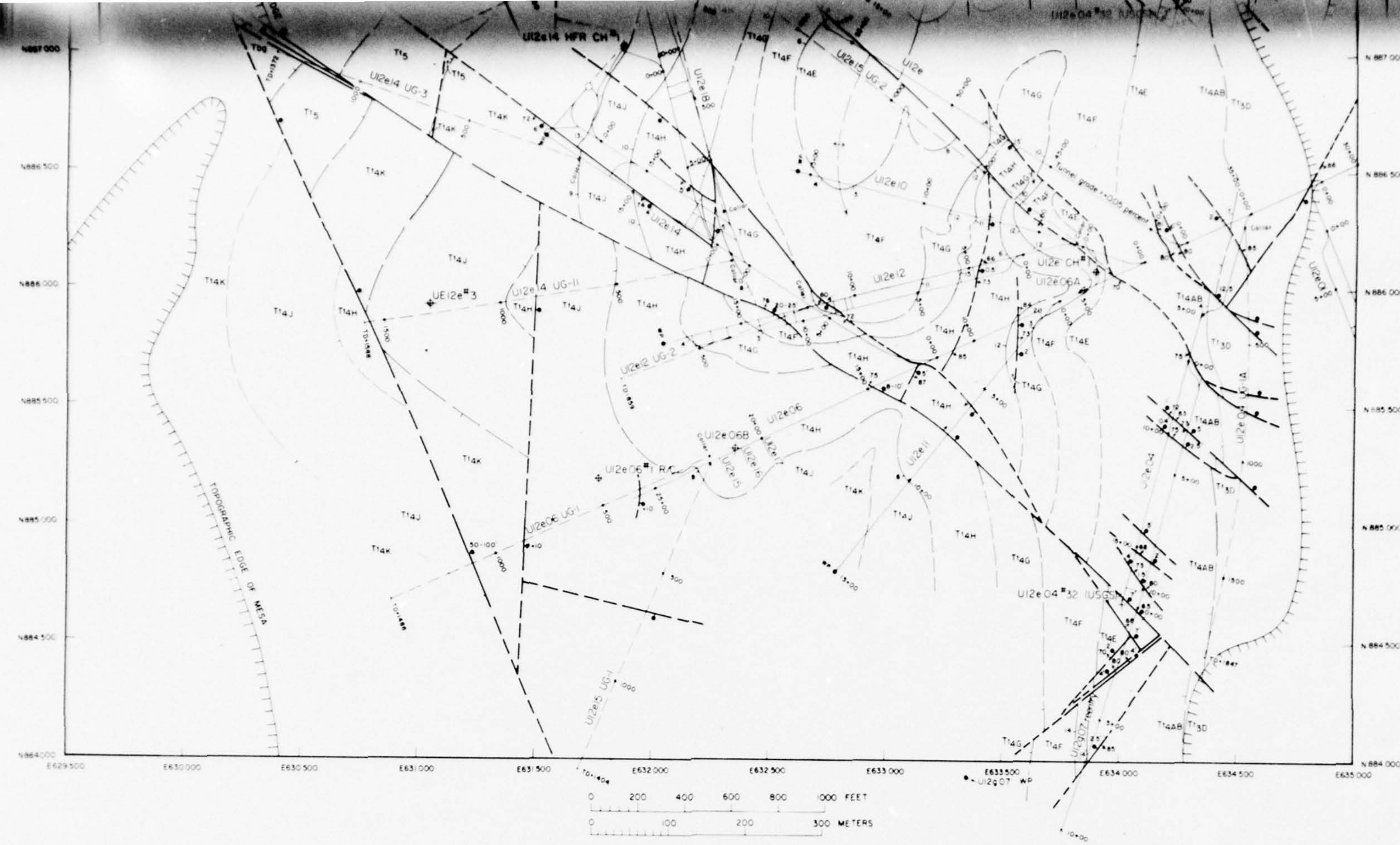

PLATE 4. TUNNEL-LEVEL GEOLOGIC MAP OF THE U12e TUNNEL COMPLEX. AVERAGE WORKING POINT ELEVATION IS 6,168 FEET.

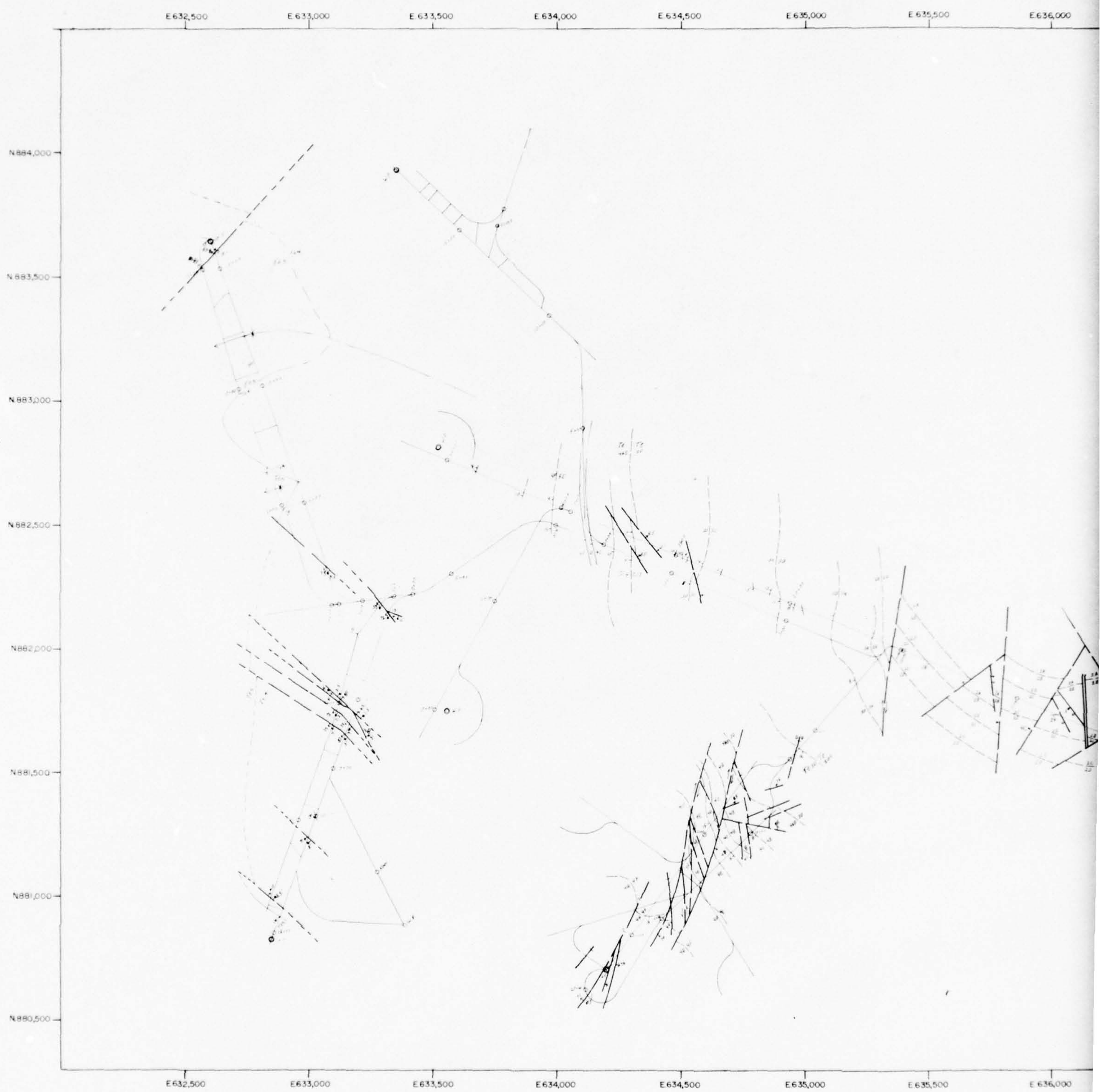

PLATE 5. TUNNEL LEVEL GEOLOGIC MAP OF THE U12g TUNNEL COMPLEX.

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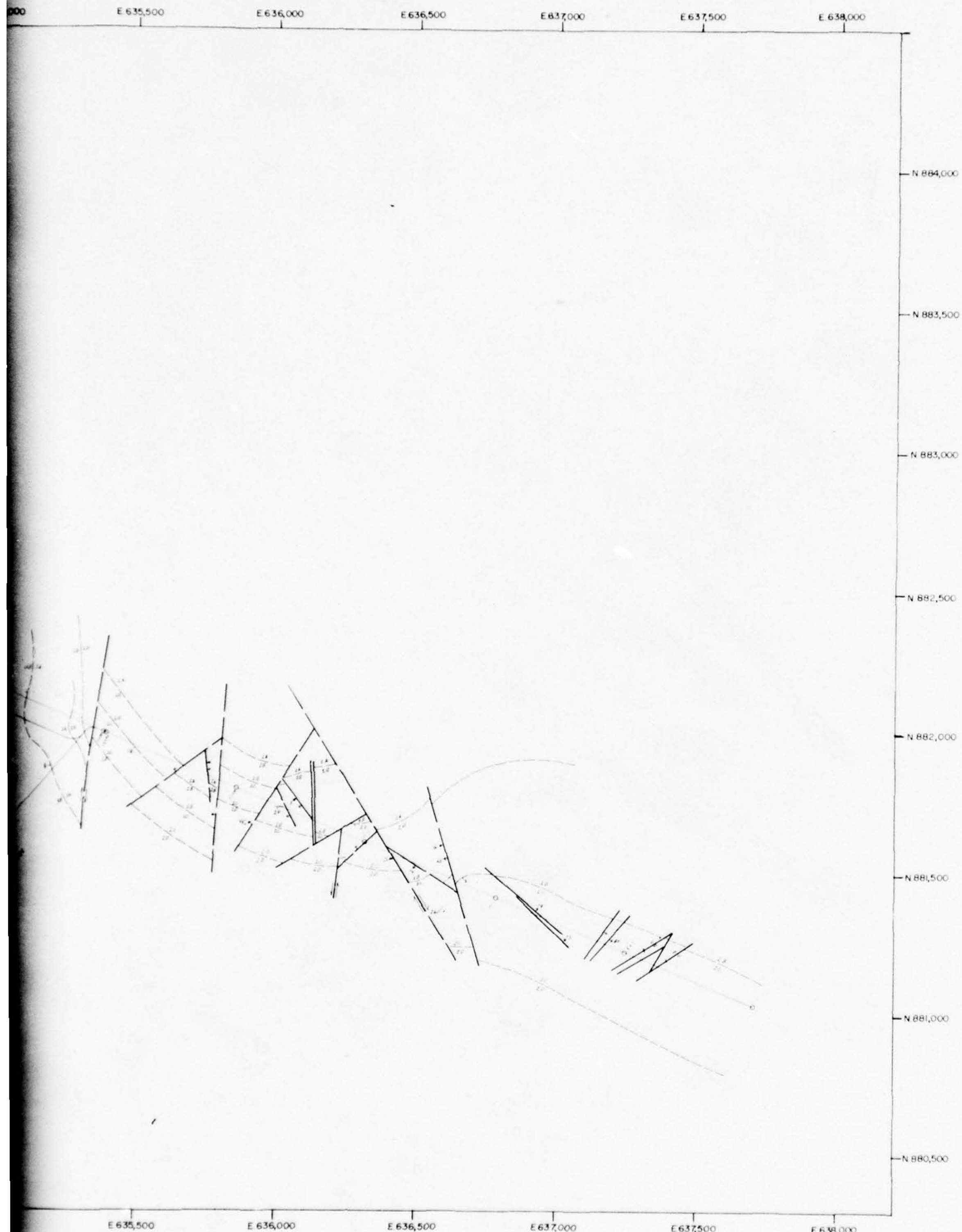

E 635,500
E 636,000
E 636,500
E 637,000
E 637,500
E 638,000
N 884,000
N 883,500
N 883,000
N 882,500
N 882,000
N 881,500
N 881,000
N 880,500

2

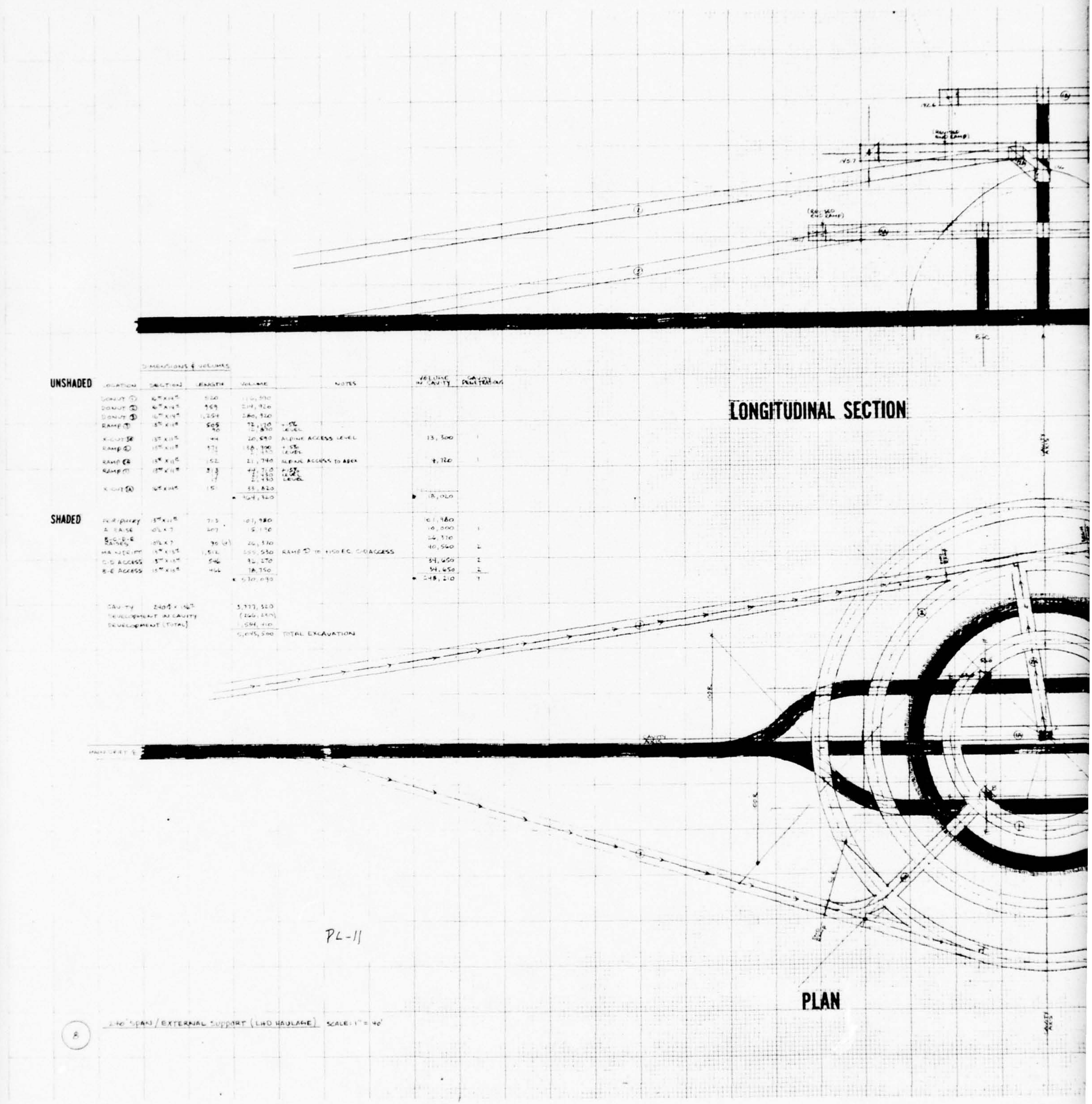
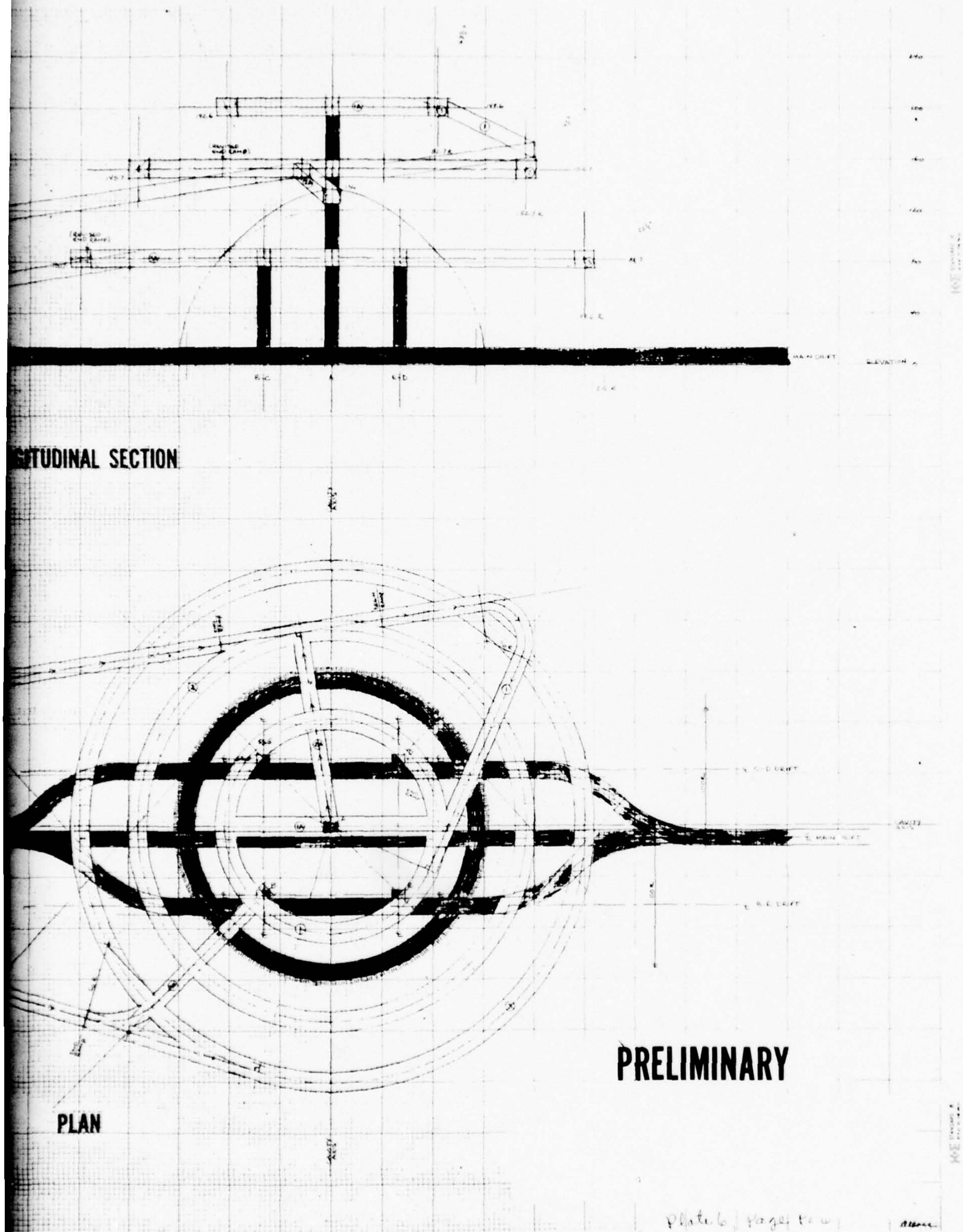


PLATE 6. LARGE CAVITY STUDY FOR DOD—240' SPAN-EXTERNAL SUPPORT CONFIGURATION FROM DONUT DRIFTS 1, 2 & 3.



GITUDINAL SECTION

PLAN

PRELIMINARY

Plate 6, page two

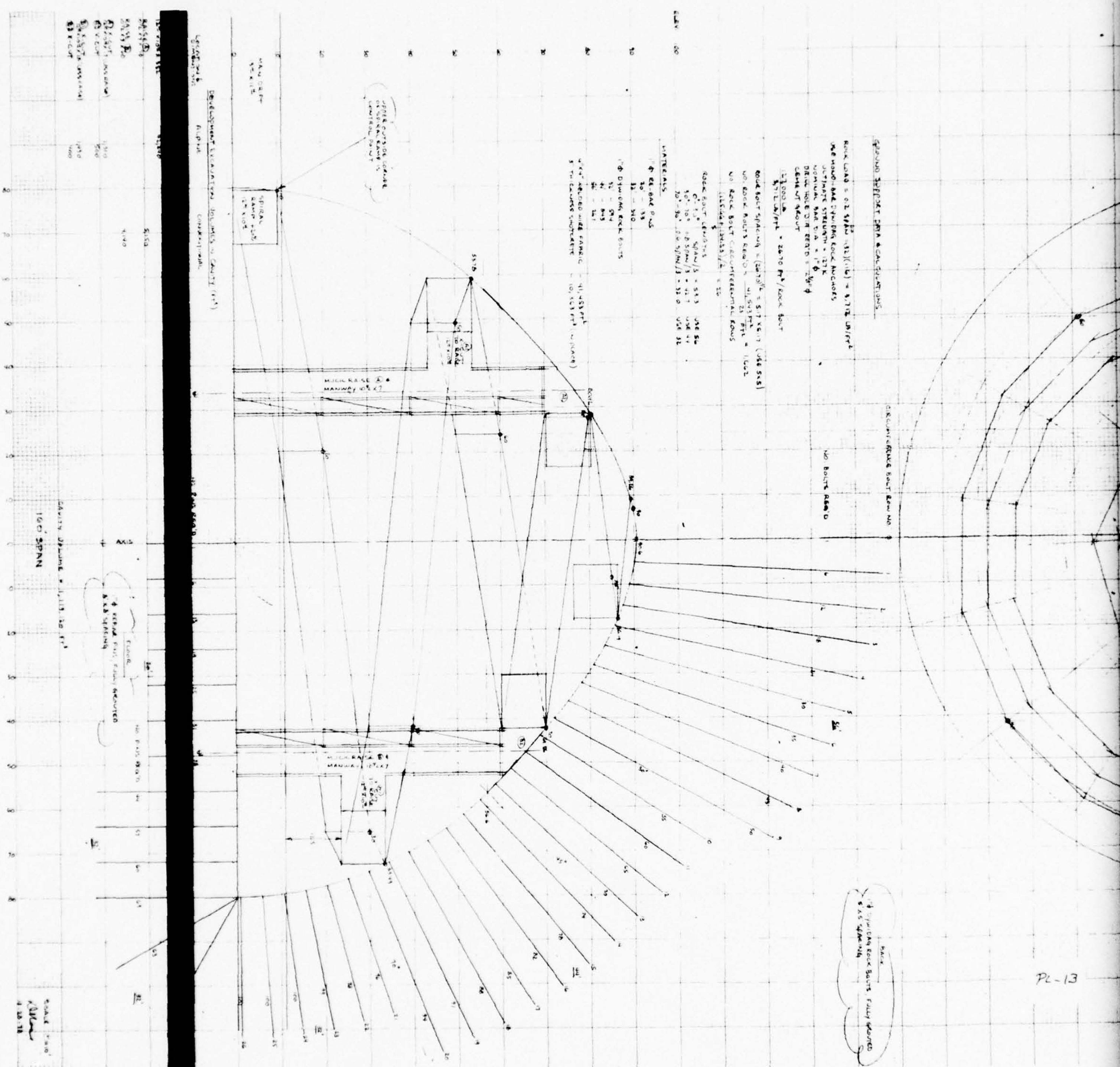
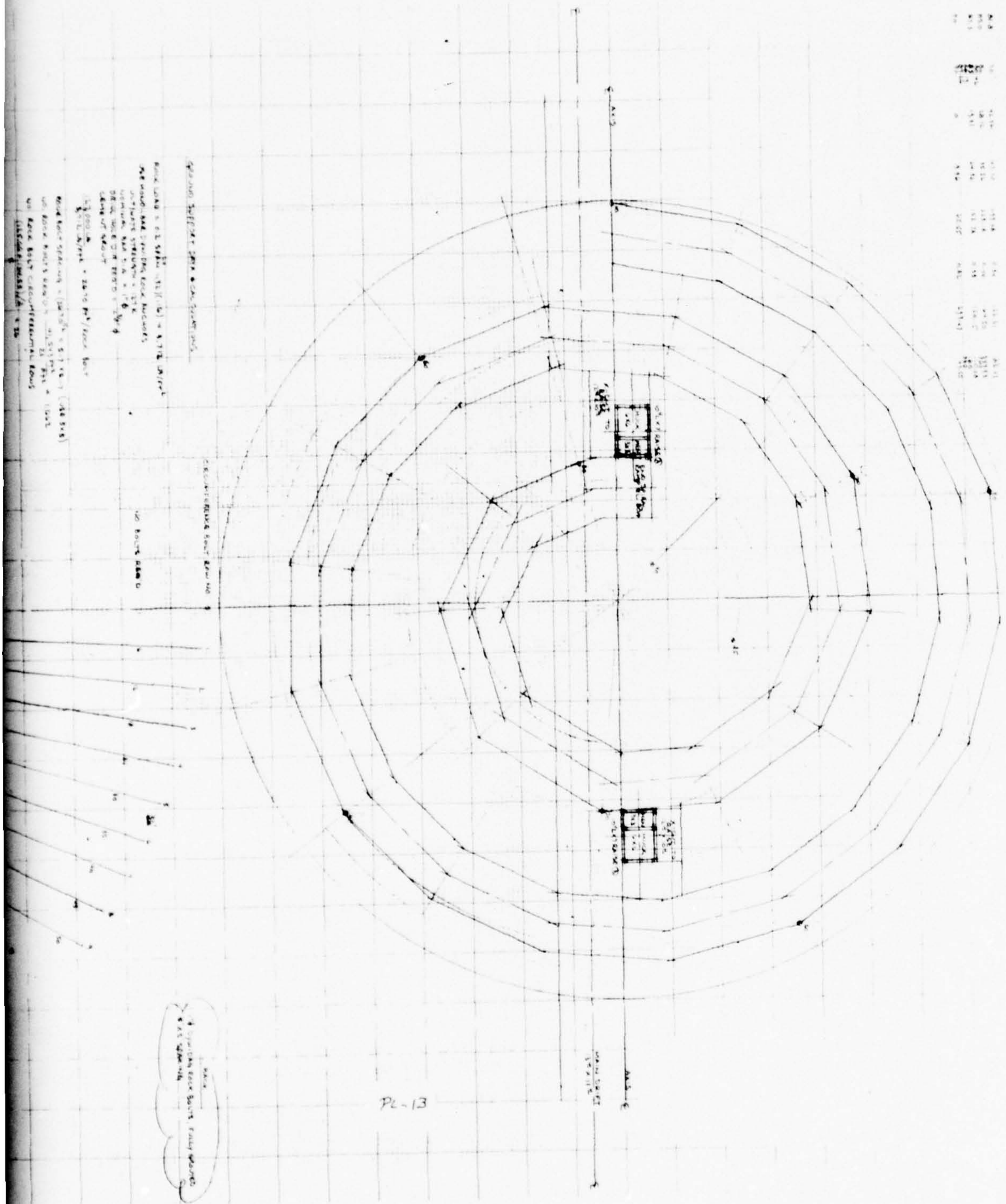


PLATE 7. LARGE CAVITY STUDY FOR DOD—160' SPAN-INTERNAL SUPPORT ALPINE/LONG HOLE DRILL-BLAST.



PL-13

APPENDIX A

LITHOLOGIC LOGS OF DRILL HOLES

U12e#1 Vertical

U12e.15 UG-2

U12e.14 UG-10

LITHOLOGIC LOG OF THE UE12e#1 VERTICAL DRILL HOLE

Stratigraphic and Lithologic Description	Thickness of Interval Meters (feet)	Depth to Bottom of Interval Meters (feet)
No core (cased off)	60.9 (200)	60.9 (200)
Timber Mountain Tuff		
Rainer Mesa Member		
Tuff, ash-flow, light-brownish-gray, pale-red, and pinkish-gray, densely welded to partially welded	24.0 (79)	85.0 (279)
Tuff, ash-flow, slight welded to nonwelded	21.6 (71)	106.6 (350)
Paintbrush Tuff		
Tuff, calc-alkaline ash-fall, tuffaceous sandstone, and some peralkaline ash-fall, reworked ash-fall, pale-gray to moderate-brown, thin- to thick-bedded; contains some large pumice; contains slight amounts of argillization; totally unconsolidated; incompetent, sandy textured	212.1 (696)	318.8 (1,046)
Belted Range Tuff		
Grouse Canyon Member		
Tuff, peralkaline ash-flow, densely welded, moderate-brown to dusky-brown, massive, crystal-rich	5.4 (18)	324.3 (1,064)
Tunnel bed 5		
Tuff, peralkaline ash-fall, grayish-yellow, massive, zeolitized, slightly argillized, partially competent to incompetent	22.8 (75)	347.1 (1,139)
Tunnel bed 4K		
Tuff, calc-alkaline ash-fall, peralkaline ash-fall, and minor amounts of reworked ash-fall, grayish-yellow to moderate-reddish-brown, zeolitized, thick-bedded; at 363 to 363.9 m (1,191-1,194 ft) highly argillized, silicified at 365.1 m (1,198 ft), thin argillized beds scattered throughout peralkaline zones, partially incompetent	34.7 (114)	381.9 (1,253)
Tunnel bed 4J		
Tuff, calc-alkaline ash-fall, and peralkaline ash-fall, moderate-redish-brown and grayish-yellow, massive; contains thin to thick beds of peralkaline ash-fall tuff, prominent grayish-yellow to grayish-orange-pink pumice throughout interval; zeolitized; some slightly argillized zones; fairly competent	16.4 (54)	398.3 (1,307)
Tunnel bed 4H		
Tuff, calc-alkaline ash-fall, reworked ash-fall, and some peralkaline ash-fall, grayish-yellow with minor amounts of moderate-reddish-brown, thin- to thick-bedded; zeolitized, argillized at 404.7 m (1,328 ft), lithic fragment clustering at 404.7 m (1,328 ft) fairly competent	11.5 (38)	409.9 (1,345)

LITHOLOGIC LOG OF THE UE12e#1 VERTICAL DRILL HOLE-- Continued

Stratigraphic and Lithologic Description	Thickness of Interval Meters (feet)	Depth to Bottom of Interval Meters (feet)
Tunnel bed 4G		
Tuff, calc-alkaline ash-fall, minor reworked ash-fall, yellowish-gray, minor moderate-reddish-brown, thick-bedded; zeolitized, large lithic fragments, salt-and-pepper texture in lower portion of unit; (0.5 ft) tuffaceous siltstone at bottom of unit marks 4F-4G contact; fairly competent ----------	7.0 (23)	416.9 (1,368)
Tunnel bed 4F		
Tuff, calc-alkaline ash-fall and reworked ash-fall, yellowish-gray to grayish-yellow, minor lenses of moderate reddish-brown, thin-bedded; zeolitized, thin lenses of siltstone scattered in upper portion of subunit, most of unit has salt-and-pepper texture; fairly competent -------	8.8 (29)	425.8 (1,397)
Tunnel bed 4E		
Tuff, calc-alkaline ash-fall, moderate-reddish-brown, massive; zeolitized, prominent cobble-size lithic fragments at 431.2-431.9 m (1,415-1,417 ft); fault at 425.8 m (1,397 ft); fairly competent ----------	7.3 (24)	433.1 (1,421)
Tunnel bed 4CD		
Tuff, calc-alkaline ash-fall, reworked ash-fall, tuffaceous sandstone, minor thin beds of tuffaceous siltstone, yellowish-gray, grayish-yellow, minor amounts of moderate-reddish-brown, thin-bedded, zeolitized, salt-and-pepper texture, silicified at 442 m (1,450.4 ft) and 446.5 m (1,465 ft), siltstone at 439.6 m (1,442.3 ft); fairly competent ----------	13.4 (44)	446.5 (1,465)
Tunnel bed 4AB		
Tuff, calc-alkaline ash-fall, reworked ash-fall, tuffaceous sandstone, minor peralkaline ash-fall, grayish-orange pink, dark-reddish-brown, thin- to thick-bedded, zeolitized, some thin silicified lenses; fairly competent ----------	11.5 (38)	458.1 (1,503)
Tunnel bed 3D		
Tuff, calc-alkaline ash-fall and minor reworked ash-fall, grayish-orange-pink, dusky-red, massive, zeolitized; faint bedding in lower portion of subunit; competent ----------	14.3 (47)	472.4 (1,550)

LITHOLOGIC LOG OF THE UE12e#1 VERTICAL DRILL HOLE-- Continued

Stratigraphic and Lithologic Description	Thickness of Interval Meters (feet)	Depth to Bottom of Interval Meters (feet)
Tunnel bed 3 BC		
Tuff, Calc-alkaline ash-fall, reworked ash-fall, and tuffaceous sandstone, grayish-pink, pale-red, moderate-reddish-brown, thin- to thick-bedded, zeolitized; mottled appearance; contains very minor lithic fragments in upper part of unit; fairly competent ----------	38.1 (125)	510.5 (1,675)
Tunnel bed 3A		
Tuff, calc-alkaline ash-fall with some reworked ash-fall, and tuffaceous sandstone in lower part of the subunit, grayish-red to pale-red, massive, zeolitized, bedded in lower part; contains black minerals with haloes; contains few lithic fragments; competent -------	16.4 (54)	526.9 (1,729)
Belted Range Tuff		
Tub Spring Member		
Tuff, peralkaline ash-flow, grayish-yellow-green, moderate-reddish-brown, grayish-yellow to light-olive, partially welded, flattened pumice; fairly competent ----------	6.4 (21)	533.4 (1,750)
Tunnel bed 2		
Tuff, calc-alkaline ash-fall, reworked ash-fall, and tuffaceous sandstone, moderate-reddish-brown, grayish-yellow, yellowish-gray, and light brown, thin-to thick-bedded, zeolitized; some silicified beds, interval with pisolites included; fairly competent to competent -----	39.3 (129)	572.7 (1,879)
Crater Flat Tuff		
Tuff, ash-flow, very dark red to grayish-red-purple, massive, very densely welded, flow structure obvious, contains abundant equi-granular quartz, feldspar and biotite; contains biotitie in pumice; competent ----------	1.5 (5)	574.2 (1,884)
Tunnel bed 1		
Tuff, reworked calc-alkaline ash-fall and tuffaceous sandstone, yellowish-gray, grayish-orange-pink, moderate-reddish-brown and pale- red, thin- to thick-bedded, zeolitized, predominantly fine-grained; some thin silicified beds; competent ----------	30.4 (100)	604.4 (1,983)

LITHOLOGIC LOG OF THE UE12L#1 VERTICAL DRILL HOLE -- Continued

Stratigraphic and Lithologic Description	Thickness of Interval Meters (feet)	Depth to Bottom of Interval Meters (feet)
Tertiary older tuff (probably Crater Flat Tuff or Red Rock Valley Tuff)		
Tuff, ash-flow, pale-red with scattered blotches of light-red, grayish-pink, massive, flow structure obvious, crystal-rich with particularly quartz and biotite; competent --------------------------------	2.4 (8)	606.8 (1,991)
Tunnel bed 1		
Tuff, reworked calc-alkaline ash-fall and tuffaceous sandstone, yellowish-gray, grayish-orange-pink, moderate-reddish-brown and pale-red, thin to thick-bedded, zeolitized, predominantly fine-grained; some thin silicified beds; competent -----------------------------	2.7 (9)	609.6 (2,000)
Total depth --------------		609.6 (2,000)

LITHOLOGIC LOG OF THE U12e.15 UG-2 DRILL HOLE

Stratigraphic and lithologic description	Thickness of Interval Meters (feet)	Depth to Bottom of Interval Meters (feet)
Tunnel beds		
Tunnel bed 4F		
Tuff, ash-fall and minor amounts of reworked ash-fall tuff, very pale orange, pale-grayish-orange and yellowish-gray, zeolitized, thick bedded to massive, competent; small fault at 7 m (23 ft) downthrown on collar side; pale-reddish-brown splotches in rock on either side of fault ----------	80.1 (263)	80.1 (263)
Tuff, peralkaline ash-fall tuff, pale-greenish-yellow, zeolitized, thin-bedded, competent ----------	2.7 (9)	82.9 (272)
Tuff, reworked ash-fall tuff, ash-fall tuff, and tuffaceous sandstone, yellowish-gray, zeolitized with moderate-orange-pink, silicified at bottom of interval, thin-bedded, competent to slabby ----------	18.5 (61)	101.4 (333)
Tunnel bed 4G		
Tuff, ash-fall and tuffaceous sandstone, pale-brown, zeolitized, thin-bedded, competent, with very pale orange and yellowish-gray pumice ------	2.1 (7)	103.6 (340)
Tuff, peralkaline ash-fall tuff, yellowish-gray, zeolitized, massive with faint layering, competent ----------	10.9 (36)	114.6 (376)
Tunnel bed 4H		
Tuff, reworked ash-fall and ash-fall tuff, pale-greenish-yellow with moderate-red streaks and moderate-red in lower half of interval, zeolitized, thin-to thick-bedded, competent ----------	14.9 (49)	129.5 (425)
Fault zone, downthrown on collar side of fault, no core recovered ----------	3.9 (13)	133.5 (438)
Tunnel bed 4G		
Tuff, peralkaline ash-fall, repetition of 103.6-114.6 m (340-376 ft) interval; stratigraphy below this interval indicates a syncline in this interval ----------	9.7 (32)	143.2 (470)
Fault zone, downthrown on collar side of fault, no core recovered ----------	2.7 (9)	145.9 (479)

Stratigraphic and Lithologic Description	Thickness of Interval Meters (feet)	Depth to Bottom of Interval Meters (feet)
Tunnel beds - Continued		
Tunnel bed 4G		
Tuff, ash-fall, repetition of upper part of 101.4-103.6 m (333-340 ft) interval	0.9 (3)	146.9 (482)
Tunnel bed 4F		
Tuff, reworked ash-fall, ash-fall tuff and tuffaceous sandstone, repetition of 82.9-101.4 m (272-333 ft) interval, pale reddish-brown streaks in upper half of interval; silicified beds less than 0.3 m (1 ft) thick at 146.9, 153, 155.1, and 156.9 m (482, 502, 509, and 515 ft)	19.2 (63)	166.1 (545)
Tuff, ash-fall and reworked ash-fall tuff, repetition of 0-80.1 m (0-263 ft) interval; pale reddish brown between 185.9 and 189.2 m (610 and 621 ft); small fault at 183.1 m (601 ft) upthrown on collar side of fault	38.2 (93)	194.4 (638)
Tunnel bed 4E		
Tuff, ash-fall and minor amounts of reworked ash-fall tuff, moderate-reddish-brown to grayish-orange-pink with yellowish-gray to very pale orange pumice, zeolitized, massive with a few thin beds, competent	27.1 (89)	220.9 (725)
Tunnel beds 4A-D		
Tuff, ash-fall, reworked ash-fall tuff and minor amounts of tuffaceous sandstone, yellowish-gray, pale-greenish-yellow and dusky-yellow, zeolitized with a few argillized or silicified beds less than 0.6 m (2 ft) thick, very thin to thick-bedded competent; sandstone and reworked intervals at 220.9-222.8, 227.3-281.3, 328.5-359.9, and 378.2-382.2 m (725-731, 910-923, 1,078-1,131, and 1,241- 1,254 ft) are tunnel bed 4CD, but lack of typical red beds in tunnel bed 4AB make contact uncertain, synclinal axis at 279.1 m (916 ft); anti-clinal axes at 238.9 and 363 m (784 and 1,191 ft)	161.2 (529)	382.2 (1,254)

LITHOLOGIC LOG OF THE U12e.15 UG-2 DRILL HOLE - Continued

Stratigraphic and Lithologic Description	Thickness of Interval Meters (feet)	Depth to Bottom of Interval Meters (feet)
Tunnel beds - Continued		
Tunnel bed 4E		
Tuff, ash-fall with minor amounts of reworked ash-fall tuff and tuffaceous sandstone, moderate-reddish-orange to pale-reddish-brown, zeolitized, thick-bedded; contains scattered large (to 50 mm) volcanic lithic fragments in first 6 m (20 ft) of interval	29.8 (98)	412.0 (1,352)
Tunnel bed 4F		
Tuff, ash-fall and minor amounts of reworked ash-fall tuff, yellowish-gray to pale-greenish-yellow, same as 0-80.1 m (0-263 ft) interval	13.7 (45)	425.8 (1,397)
Tuff, peralkaline ash-fall, pale-greenish-yellow, same as 80.1-82.9 m (263-272 ft) interval	2.1 (7)	427.9 (1,404)
Tuff, reworked ash-fall, ash-fall tuff, and tuffaceous sandstone, yellowish-gray, same as 82.9-101.4 m (272-333 ft) interval; silicified beds less than 0.2 m (1 ft) thick at 430.9, 438.3-439.5, and 473-474.5 m (1,414, 1,438-1,442, and 1,552-1,557 ft)	46.6 (153)	474.5 (1,557)
Tunnel bed 4G		
Tuff, ash-fall and reworked ash-fall tuff, pale-greenish-yellow to pale-brown, same as 101.4-103.6 m (333-340 ft) interval	6.7 (22)	481.2 (1,579)
Tuff, peralkaline ash-fall, yellowish-gray to pale-greenish-yellow, same as 103.6-129.5 m (340-425 ft) interval	28.3 (93)	509.6 (1,672)
Tunnel bed 4H		
Tuff, ash-fall and reworked ash-fall tuff with minor amounts of peralkaline ash-fall tuff and tuffaceous sandstone, greenish-yellowish-gray, pale-brown, and pale-to moderate-reddish-brown, zeolitized, thin-bedded to very thin bedded, competent	66.1 (217)	575.7 (1,889)
Tunnel bed 4J		
Tuff, ash-fall and peralkaline ash-fall tuff with minor amounts of reworked ash-fall tuff, moderate-reddish-brown to moderate-reddish-orange and greenish-yellow to dusky-yellow, zeolitized to partly argillized, thin- to thick-layering, competent to poorly competent; visible argillization between 589.9 and 617.2 m (1,965 and 2,025 ft)	63.0 (207)	669.3 (2,196)

Stratigraphic and lithologic Description	Thickness of Interval Meters (feet)	Depth to bottom of Interval Meters (feet)
Tunnel beds - continued		
Tunnel bed 4K		
Tuff, ash-fall and peralkaline ash-fall tuff, yellowish-gray and moderate-reddish-brown, zeolitized, thin- to mostly thick bedded, competent; synclinal axis at 684.2 m (2,245 ft)	29.8 (98)	669.2 (2,294)
Tunnel bed 4J		
Tuff, ash-fall with minor amounts of reworked ash-fall tuff, moderate-redish-brown to moderate-reddish-orange, same as 575.7-669.3 m (1,889-2,196 ft) interval, competent to poorly competent; visible argillization between 720.8 and 746.7 m (2,365 and 2,450 ft) ------------	62.7 (206)	762.0 (2,500)
Total depth ---------------		762.0 (2,500)

LITHOLOGIC LOG OF THE U12e.14 UG-10 DRILL HOLE

Stratigraphic and Lithologic Description	Thickness of Interval Meters (feet)	Depth to Bottom of Interval Meters (feet)
Tunnel beds		
Tunnel bed 4G		
Tuff, peralkaline(?) ash-fall, very pale orange, zeolitized to slightly argillized, massive, competent to poorly competent -------------	50.5 (166)	50.5 (166)
Tunnel bed 4H		
Tuff, ash-fall and reworked ash-fall tuff, yellowish-gray, pale-yellowish-brown, and grayish-orange-pink, zeolitized to partly argillized, thin-to very thin-bedded, poorly competent; faults at approximately 55.1 and 65.5 m (181 and 215 ft) are displaced down to collar less than 3 m (10 ft) ----------------------------------	20.1 (66)	70.7 (232)
Tuff, ash-fall, with thin beds of reworked ash-fall and peralka-line ash-fall, light-red and pale-to moderate-reddish-brown to 85.3 m (280 ft), and very pale orange, grayish-orange-pink, and pale-yellowish-brown with light- to moderate-red blotches below 85.3 m (280 ft), zeolitized, thin- to thick-bedded, competent ----------	85.6 (281)	156.3 (513)
Tuff, peralkaline ash-fall and reworked peralkaline ash-fall tuff and reworked ash-fall tuff, yellowish-gray, grayish-orange, and light-red, zeolitized with thin silicified beds, thin- to very thin bedded, competent to very competent --------------------------	51.2 (168)	207.5 (681)
Tunnel bed 4J		
Tuff, ash-fall and peralkaline ash-fall tuff, moderate-to pale-reddish-brown and pale-yellowish-brown to yellowish-gray, zeolitized; massive thick layers with lenses of peralkaline material; competent --	114.3 (375)	321.8 (1,056)
Tunnel bed 4K		
Tuff, peralkaline ash-fall and ash-fall tuff, dusky-yellow, moderate yellowish-brown, and pale-reddish-brown, zeolitized, thin bedded, competent; fault at bottom of interval displaced down to working point 3-6 m (10-20 ft) --	18.5 (61)	340.4 (1,117)

Stratigraphic and Lithologic Description	Thickness of Interval Meters (feet)	Depth to Bottom of Interval Meters (feet)
Tunnel beds - Continued		
Tunnel bed 4K - continued		
Tuff, ash-fall, reworked ash-fall tuff and peralkaline ash-fall tuff, yellowish-gray to dusky-yellow, pale-yellowish-brown, and pale-red, zeolitized, thin- to thick-bedded, competent to poorly competent; synclinal axis at approximately 347.4 m (1,140 ft) ----------	56.3 (185)	396.8 (1,302)
Repetition of 340.4- to 321.8 m (1,117- to 1,056 ft) interval ------------	5.1 (17)	402.0 (1,319)
Tunnel bed 4J		
Repetition of 321.8- to 207.5 m (1,056- to 681.0 ft) interval ------------	68.8 (226)	470.9 (1,545)
Tunnel bed 4H		
Repetition of 207.5- to 50.5 m (681- to 166 ft) interval argillized between 535.4 and 530.6 m (1,724 and 1,741 ft); faults at 482.4 and 501.3 m (1,583 and 1,645 ft) have less than 1.5 m (5 ft) of displacement down to working point ------------------------------------	81.3 (267)	549.8 (1,804)
Tunnel bed 4G		
Repetition of 50.2 m (165 ft) to collar interval ------------------------	19.8 (65)	572.1 (1,877)
Tuff, reworked ash-fall and ash-fall tuff, yellowish-gray and pale-yellowish-brown, zeolitized, thin-to very thin bedded, competent -------	5.7 (19)	577.9 (1,896)
Tunnel bed 4F		
Tuff, reworked ash-fall and ash-fall tuff, yellowish-gray, pale-yellowish-brown and pale-reddish-brown, zeolitized with scattered very thin silicified beds, thin- to very thin bedded, competent; faults at 595.5, 598.0, and 600.4 m (1,954, 1,962, and 1,970 ft) all have displacements of less than 1.5 m (5 ft) down to the collar --	32.7 (107.5)	610.6 (2,003.5)
Total depth -----------------		610.6 (2,003.5)

APPENDIX B

DEPTH OF PLASTIC ZONE AROUND A SPHERE

APPENDIX B: DEPTH OF PLASTIC ZONE AROUND A SPHERE

The depth of the plastic zone around a sphere in a medium subjected to a uniform all-around external pressure can provide an approximate estimate of the size of the fractured zone behind the curved walls of the hemispherical chamber. Such analyses should be compared with the observed fracturing in Red Hot/Deep Well caverns, where the effects of non-uniform and complex boundary conditions were observed directly.

A. Equilibrium Relation: Body forces assumed equal to zero:

$$\sigma_r \, r d\theta = (\partial\sigma_r + \sigma_r)\,(r + dr)\, d\theta + 2\sigma\theta \, d\theta dr$$

$$\frac{\partial\sigma_r}{\partial r} + \frac{\sigma_r - 2\sigma\theta}{r} = 0 \qquad (1)$$

B. Limiting equilibrium: The following relation is assumed in the plastic zone:

$$\sigma\theta = \sigma_r N\phi + q_u \qquad (2)$$

where q_u = unconfined compressive strength

N_ϕ = $\tan^2(45 + \phi/2)$; ϕ = angle of internal friction

C. Substitute (2) into (1) to obtain relation for the radial stress in the plastic zone:

$$\frac{\partial\sigma_r}{\partial r} = \frac{\sigma_r (2 N\phi - 1) + 2q_u}{r}$$

$$\sigma_r (2 N\phi - 1) + 2 q_u = Cr^{(2 N\phi - 1)}$$

Evaluate C: when $r = a$, $\sigma_r = P_i$

where P_i is the internal pressure in the spherical cavity.

$$\frac{\sigma_r\,(2\,N\phi\;-1) + 2q_u}{P_i\,(2\,N\phi\;-1) + 2q_u} = \left(\frac{r}{a}\right)^{(2\,N\phi\;-1)} \tag{3}$$

D. Determine R, the radius of the plastic zone.

In the elastic zone; the following condition applies:

$$\sigma_r + 2\sigma\theta = 3\,P_o \tag{4}$$

where P_o is the external pressure applied to the medium at $r \to \infty$. In the plastic zone, equation 2 applies. The two equations can be equated to give the condition at the boundary between the elastic and plastic zones, at $r = R$.

$$\sigma_R = \frac{3\,P_o - 2\,q_u}{1 + 2\,N\phi} \tag{5}$$

Obtain radius, R, of plastic zone by substituting (5) in (3):

$$\frac{R}{a} = \left[\frac{3P_c\,(2N\phi\;-1) + 4q_u}{(1 + 2N\phi)[P_i(2N\phi - 1) + 2q_u}\right]^{\frac{1}{2N\phi\;-1}} \tag{6}$$

To estimate the size of the fractured zone around the cavern in tuff, the following material properties are assumed:

In-situ stress: $P_o = 1000$ psi

Intact rock strength: $q_u = 1400$ psi $\qquad \phi = 30°$

From elastic theory for a spherical cavity in a uniform stress field, the tangential stress concentration at the wall of the opening is 1.5 P_o. Thus the above values will result in an elastic tangential stress of 1500 psi, slightly in excess of the intact unconfined compressive strength, so that some fracturing might be expected.

Substitution of the above values in equation (6) gives: $R/a = 1.01$ (equivalent to a plastic zone 1.5 ft thick around a 300-ft diameter cavern).

Because the horizontal and vertical in-situ stresses are not equal, higher stress concentrations will develop than those predicted for a uniform stress, P_o. In addition, the shape of the cavern during initial excavation stages will not approximate a sphere, but will be closer to a flat ellipsoid. In this case, the stress concentrations near the base of the wall may be greater and more fracturing may develop. These effects might be approximated in the above solution by increasing P_o by a percentage equivalent to the increased stress concentrations that could develop. Assuming a 25% increase in P_o to account for stress concentrations:

$$P_o = 1250$$

$$q_u = 1400 \text{ psi}$$

$$\phi = 30°$$

then; substituting in equation 6: $R/a = 1.045$, (equivalent to a plastic zone 6.75 ft thick around a 300-ft-diameter cavern).

The above depths of the fractured zone are consistent with the values predicted in Chapter 4, based on previous observations in Red Hot/Deep Well and on the expected influence of the proposed construction procedures for the large caverns. Fracturing and slabbing in the floor of the chamber will extend to greater depths than those predicted for the walls.

APPENDIX C

MEASUREMENT OF DISPLACEMENTS IN TUNNELS

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MEASUREMENT OF DISPLACEMENTS IN TUNNELS

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SUMMARY:

Displacement measurements made with borehole extensometers and inclinometers are the primary means of monitoring the stability of a tunnel or underground chamber in rock or soil. For tunnels in soil, they also provide a means of assessing the movements which could damage nearby structures. Examples of measured rock and soil displacements around tunnels are presented. Criteria for evaluating the stability of an underground chamber from displacement measurements are summarized.

RÉSUMÉ

Des mesures de déplacement faites avec extensomètre et avec inclinometre sont la premiere méthode en surveillant la stabilité d'un tunnel ou passage souterrain dans la roche ou dans la terre. Pour les tunnels dans la terre, ils fournient aussi une méthode pour évaluer les mouvements qui pourraient damager les structures tout près. Des exemples de roche mésurée et de déplacements de terre autour des tunnels sont presentés. Criteria pour évaluer la stabilité d'un passage souterrain d'après les mésures de déplacement sont resumés sommairement.

1. INTRODUCTION

The measurement of displacement using borehole instruments has proven to be one of the most reliable means of evaluating and monitoring the performance of an underground opening in rock or soil. Such measurements should be considered an extension of the visual observations and geologic mapping normally conducted in a tunnel during construction.

Displacement measurements sum the behavior of the rock or soil mass over some measurement interval, thus they do not have the extreme variability which usually results when attempting to measure a quantity, such as strain or pressure, at a point. By measuring displacements at the tunnel wall and at several radial distances beyond the tunnel wall it is possible to determine if the excavation and support of the tunnel are having a local or widespread influence on displacements, and to relate the events in the tunnel to the ground movements away from the tunnel. The severity of the settlement or stability problem can then be assessed.

Instruments most commonly used to measure displacements are the borehole extensometer, for measuring displacements parallel to the borehole axis, and the inclinometer, for measuring displacements perpendicular to the borehole axis. Their

characteristics are summarized in the following section. Later sections of the paper present the results of some displacement measurements around tunnel and chambers in rock and soil, and describe criteria for evaluating displacement measurements.

2. CHARACTERISTICS OF BOREHOLE INSTRUMENTS

Borehole instruments should be simple, reliable units which require a minimum amount of field adjustment and which can withstand the tunnel and borehole environment without deterioration or drift. If an electronic unit must be used, there should be opportunity to recalibrate and replace the unit without losing the zero. Readings are often difficult to interpret, even when the real rock or soil performance is being measured. Thus, the further complication of non-linear calibration, large hysteresis, or drifting, erratic measurements due to faulty mechanical or electrical systems can make the results of an observation program almost unintelligible.

Borehole inclinometers are commonly used to measure lateral displacements (perpendicular to the axis of the borehole). The inclinometer consists of a torpedo which rides in a grooved casing and measures the inclination from the vertical of the cased borehole at various points in the casing. The lateral displacement at a given elevation is obtained by summing the inclinations from a fixed point, usually at the bottom of the casing.

Some of the more recently developed inclinometer systems utilize servo-accelerometers to measure inclination. Approximately four years of field experience with servo-accelerometer-type inclinometers indicates that they are accurate to slopes of 1 in 10,000 or better (±1 mm lateral displacement over a 10-meter length of casing). The fixed borehole inclinometer system, more recently developed, has a repeatability on the order of 1 in 50,000. In the past, inclinometers have primarily been used to measure lateral displacements in soil; however, they are now becoming precise enough to measure rock displacements surrounding a tunnel. Although the sensing element of the inclinometer torpedo is an electronic unit which requires maintenance and periodic adjustment and repair, it does possess the advantage that the zero readings are not lost when the torpedo is repaired or replaced.

For measuring longitudinal displacements in a borehole, extensometers with anchors at several depths in a borehole are commonly used. The most reliable borehole extensometers consist of a rod anchored in the borehole and read at the collar of the hole with a depth micrometer. Such units can be read and are repeatable to ±0.025 mm. Extensometers consisting of tensioned wires suffer significant hysteresis in long holes (greater than 10 m) due to friction between the wire and the casing or the wire and the guides, and are therefore not as accurate as the rod system (Hedley, 1969).

For evaluating the stability of tunnels in rock, displacement measurements should be made to a precision of ±0.05 mm. In soil, displacement measurements on the order of ±0.5 mm are usually adequate. For evaluating movements which could damage adjacent structures, borehole extensometers (or settlement points) may be tied to surface surveys, and survey accuracy of approximately ±2 mm may be sufficient.

3. DISPLACEMENTS IN ROCK AND SOIL

Displacement measurements have typically been used for different purposes in soil tunnels than in rock tunnels. In rock tunnels, displacement measurements are most useful in evaluating the stability of the opening. Such measurements are of greatest benefit in large chambers opened in stages, where it is possible to observe the movements taking place throughout all excavation stages and, if necessary, adjust excavation and support procedures on the basis of observations during the initial phases.

Where large, multi-stage chambers are excavated in soil, or a combination of soil and rock, displacement measurements should also be used to monitor stability. But, in most conventional soft ground tunnels, the primary use of displacement measurements is in evaluating the ground movements which could damage adjacent or overlying

utilities and structures. Such damage can occur in both small and large tunnels driven in soil, even when no stability problem exists in the tunnel.

3.1 Instrumentation for a Soil Tunnel

Surface surveys are commonly performed above a soft ground tunnel to evaluate settlement, but measurements in boreholes in the soil mass surrounding the tunnel provide much additional information. Such measurements are useful for two reasons. First, measurements immediately adjacent to the tunnel help determine the source of lost ground and relate the movement to some specific aspect of the tunneling process. Corrections to the tunneling process can then be made on the basis of the observed movements. Second, measurements of soil movement in the mass surrounding the tunnel establish the distribution and magnitude of displacement. Both lateral and vertical displacements are of interest, because either component can result in damage to adjacent or overlying structures.

Figure 1 illustrates the use of a borehole extensometer for determining the source of settlements as a tunnel shield is advanced in soft ground on the Washington, D. C. Metro. Vertical soil displacements were measured with borehole extensometers referenced to a surface survey as a 6-meter-diameter tunnel was mined in dense sands, and stiff clays beneath the extensometer location (Hansmire and Cording, 1972). The extensometer was installed from the ground surface prior to tunneling. Displacement of the anchor immediately above the tunnel crown clearly indicates that two-thirds of the soil settlement took place as the shield passed beneath the extensometer anchor, the remaining settlement took place at and behind the tail of the shield. The observations suggested that modifications to reduce lost ground over the shield, rather than at the face or at the tail of the shield, would cause the most significant reduction in soil settlement. Such a conclusion is not as easily reached by observing the surface settlements alone (Fig. 1). This conclusion subsequently proved to be correct when a modified shield, advanced through the same soil types, produced only 50 mm settlement immediately above the crown as the shield passed by, as compared to 250 mm settlement for the first shield.

To determine the distribution and magnitude of soil displacements around the Metro tunnel, a cluster of extensometers and inclinometers were installed from the ground surface in Lafayette Square prior to mining the tunnel. The measured lateral and vertical soil displacements due to tunneling are summarized in the diagram of Figure 2. The data is detailed enough to permit determination of volume changes and strains throughout the soil mass, as well as to define the limits of the significant soil displacements near the ground surface which could affect nearby structures.

3.2 Instrumentation for a Shallow Rock Chamber

Instrumentation for monitoring stability of a shallow rock chamber in rock is illustrated in Figure 3. The figure shows typical instrumentation at one of four test sections in the Dupont Circle Station, Washington, D. C. Metro. Borehole extensometers were installed as the primary instrumentation for monitoring stability during construction of the station. Multiple-position extensometers (both rod- and wire-type) were installed from the ground surface and from the pilot tunnel prior to general excavation. A complete history of the rock displacements for all stages of construction was obtained from these instruments. Double-position rod-type extensometers with expansion shell anchors were installed during construction in the crown and sidewalls of the advancing drifts, usually within 2 meters of the face. The double-position extensometers could be installed rapidly and provided flexibility to the measurement program. They were installed in positions which would have been inaccessible to instruments installed prior to excavation, or in areas where a specific geologic condition or construction condition requiring monitoring was encountered.

One of the primary concerns during construction was that the various stages be excavated and supported without excessive loosening of the rock around the excavation which would increase the permanent loads or cause instability of the relatively thin rock arch. During the initial stages, the extensometers indicated conditions where excavation and support procedures could be adjusted to minimize rock movements. They

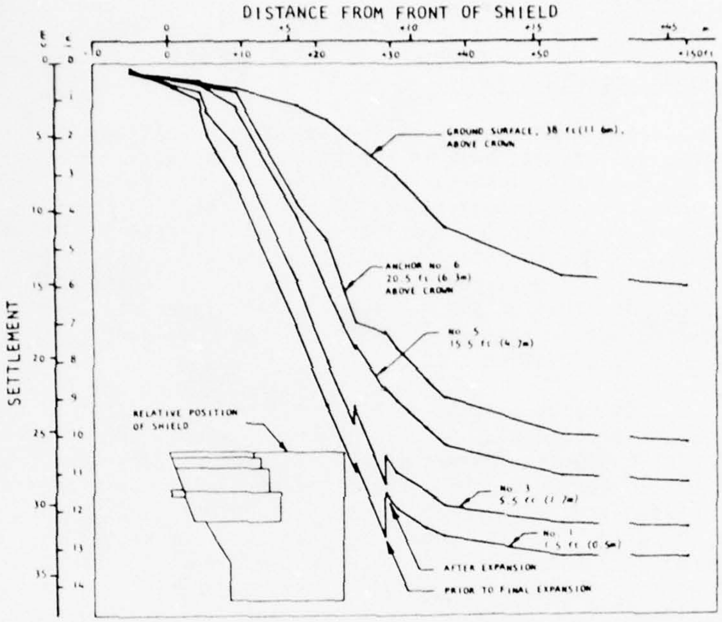

FIG. 1: EXTENSOMETER MEASUREMENTS ABOVE A SOIL TUNNEL (HANSMIRE AND CORDING, 1972).

FIG. 1: MÉSURES AVEC EXTENSOMÉTRE AU DESSUS D'UN PASSAGE SOUTERRAIN (HANSMIRE ET CORDING, 1972).

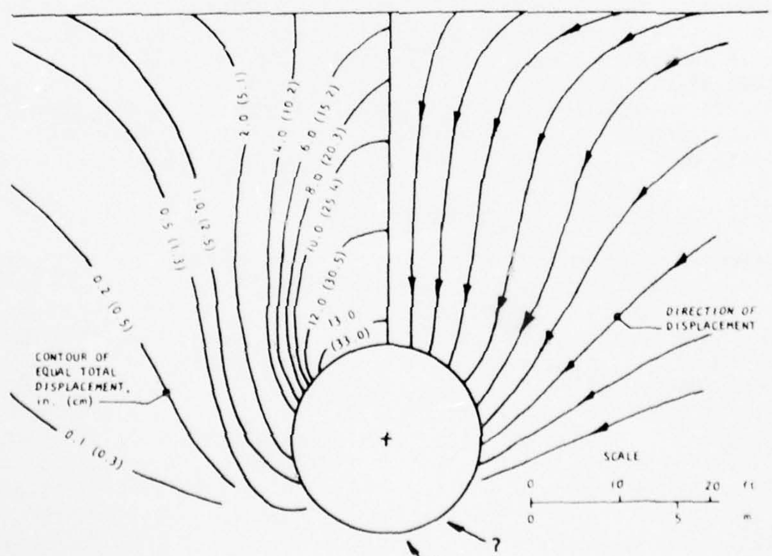

FIG. 2: PATTERN OF DISPLACEMENT AROUND A SOIL TUNNEL (HANSMIRE AND CORDING, 1972).

FIG. 2: MODÈLE DE DÉPLACEMENT AUTOUR D'UN PASSAGE SOUTERRAIN (HANSMIRE ET CORDING, 1972).

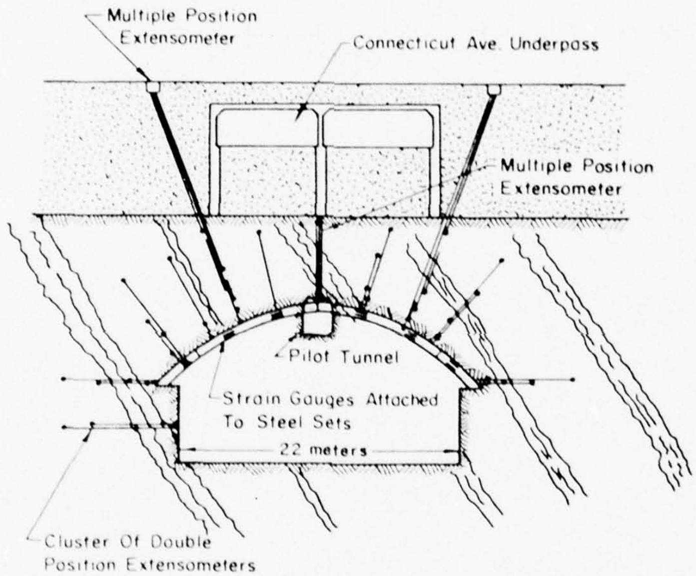

FIG. 3: TEST SECTION INSTRUMENTATION IN DUPONT CIRCLE STATION (CORDING AND DEERE, 1972).

FIG. 3: SECTION D'EXAMEN À DUPONT CIRCLE STATION (CORDING ET DEERE, 1972).

also indicated that the pre-support measures using rock bolts were successful in minimizing rock displacements as later stages were excavated. (A description of the excavation stages in the Dupont Circle Station is presented in the general report of this session.)

The performance of the final lining was monitored with the borehole extensometers as well as with strain gages in the lining. Lining displacements and loads were closely related to further excavation in the vicinity of the lining and to temperature changes in the lining.

4. CRITERIA FOR EVALUATING DISPLACEMENTS

Criteria for determining if displacements are indicative of either stable or potentially unstable behavior are outlined in this section. Displacements which indicate local instability, such as loosening of a thin rock slab, are distinguished from those which indicate a more widespread and deep-seated condition affecting the stability of the entire tunnel. In most cases, a single criterion is not adequate for evaluating the stability of a tunnel. For example, a stability criterion should not be based solely on a displacement magnitude, above which unstable conditions are assumed to exist.

Displacement measurements are most valuable when extensometers are installed at or before the beginning of excavation, and when measurements have been taken regularly throughout the entire excavation period at several locations so that a complete history of movements is available. They will be of most use if the geologic conditions and construction events in the vicinity of the measurements are also recorded and compared with the movements.

4.1 Magnitude of Displacement with Respect to Displacement Predicted from Elastic Theory

Elastic or elastic-plastic continuum solutions are quite useful for comparison with the observed displacements in a rock tunnel or chamber, even though the rock mass may suffer large displacement along joint surfaces and not behave as a continuum. The continuum solution is valuable because it provides an estimate of the displacements the mass would undergo if loosening along the joints were minimized. Unstable conditions may exist if displacements are large with respect to the displacements predicted from elastic theory.

Either a closed elastic solution, assuming simple boundary conditions, or a finite element elastic solution which approximates the more complex boundary conditions in the chamber, can be used to estimate the elastic displacements. The insitu modulus must be evaluated for use in the elastic solution. The insitu modulus can be estimated, with sufficient accuracy for predicting displacements, by reducing the laboratory modulus using a factor which accounts for the effect of jointing and weathered zones, such as the RQD (Deere, et al, 1967). For RQD values less than 60 percent, the insitu modulus is typically less than 20 percent of the laboratory modulus. Displacement measurements obtained as the excavations are initially opened can also be used to calculate the insitu modulus.

Figure 4 illustrates typical extensometer displacements observed at the Nevada Test Site in two rock bolted, 28-meter-diameter hemispherical chambers excavated in a volcanic tuff of excellent rock quality at a depth of 300 meters (Cording, 1968). Measured rock displacements are plotted versus distance from the wall of the chamber. Displacements are with respect to the anchor furthest from the wall of the opening. Positive displacements indicate rock movement toward the chamber.

Typical extensometer displacements at the wall of the cavity were on the order of 5 to 15 mm where surface slabs did not loosen extensively or where large deep-seated movements did not take place (Fig. 4a). Rock displacements decreased with distance away from the wall of the chamber. Elastic displacements were computed using both finite element and closed elastic solutions. For comparison with the extensometer displacements, the elastic displacements were computed for the increment of excavation taking

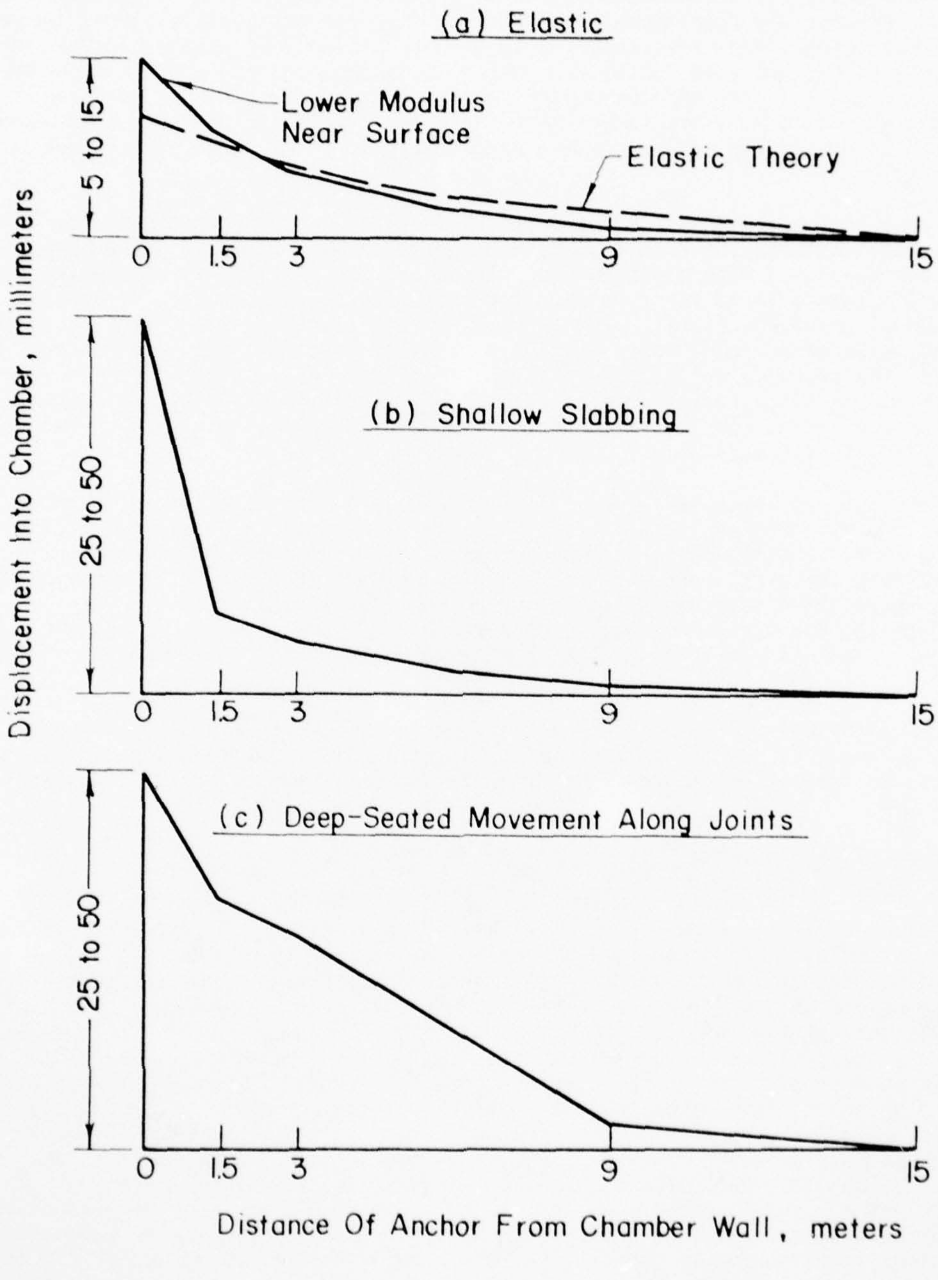

FIG. 4: TYPICAL DISPLACEMENTS IN CAVITIES I AND II, NEVADA TEST SITE (CORDING, 1968).

FIG. 4: DÉPLACEMENTS TYPIQUES DANS LES CAVITÉS I ET II (CORDING, 1968).

place after the extensometers were installed, and zero displacement was assumed at the position of the furthest extensometer anchor from the wall of the chamber. An insitu modulus equal to the laboratory modulus (35,000 kg/cm^2) was used because the excellent quality tuff was massive and almost unjointed.

The elastic displacements showed almost the same pattern and magnitude with depth as did the observed displacements. Observed displacements were slightly larger in the 1.5-meter zone immediately adjacent to the chamber wall, probably because the rock in this zone had loosened during excavation (blasting) and therefore had an insitu modulus less than the insitu modulus of the undisturbed rock (Fig. 4a). Observed displacements were 3 to 10 times larger than the computed elastic displacements where shallow slabs loosened (Fig. 4b), or where deep-seated movement along joints took place (Fig. 4c).

The extensometer displacement patterns in Figure 4 are not only typical of the two chambers in tuff, but have also been observed in many other underground openings, both large and small, in many different rock types and rock qualities. A summary of observed displacements in 13 large rock-bolted underground chambers is presented by Cording, Hendron and Deere (1971). The chambers were at depths ranging from 60 to 400 meters. Most were underground power stations. Elastic displacements were computed and compared with the reported displacements in the chambers. Where extensive rock loosening did not take place, displacements ranged from 1 to 2 times the elastic displacement computed from the insitu modulus. (In most of these cases plate load tests were used to determine the insitu modulus.)

Movement and loosening along joints were usually indicated when the observed displacements were more than 3 times the elastic displacements. In almost all cases, the investigators reporting these case histories related the large displacements to specific geologic features, such as shear zones, bedding planes, or joint systems. In many cases, where the displacements exceeded the computed elastic displacements by a factor of 5 to 10, the excavation and support procedures were modified to prevent further large movements. Most of the large displacements were on the order of 12 to 75 mm.

The magnitude of the observed displacements with respect to the computed elastic displacement is not the sole criterion for evaluating the need for modification of excavation or support procedures. Other criteria to be considered are discussed below.

4.2 Magnitude of Displacement with Respect to Measured Displacement in Well-Supported Sections of the Tunnel

Displacement measurements obtained at several locations in a tunnel can be used to establish the typical behavior of the tunnel for the given tunnel size, geologic setting, and support system. Displacements significantly greater than those previously observed may be indicative of excessive loosening of the rock surrounding the tunnel and possible stability problems.

In some cases, the typical displacements in a tunnel may exceed the calculated elastic displacements, but not be indicative of a stability problem. For example, displacements were measured with borehole extensometers in the crown of a 10-meter-wide, 7-meter-high tunnel, driven through a blocky, foliated schist at a depth of approximately 30 meters in Washington, D. C. (Mahar, Gau, Cording, 1972). The rock quality was typically fair to good and the insitu modulus was estimated to be 70,000 kg/cm^2. Because of the low stresses surrounding the opening, the computed elastic displacement due to advancement of the heading for an extensometer installed at the heading of the tunnel was only 0.25 mm. In portions of the tunnel, pattern rock bolts and shotcrete were installed near the face within a few hours after excavation. In these sections, extensometer displacements typically ranged from 0.25 to 1.0 mm. Most of the displacements were concentrated within a 1.5 meter zone beyond the surface of the tunnel, indicating that the joints bounding shallow rock blocks had opened slightly (Fig. 5a). Even though these displacements were as much as four times the calculated elastic displacement, they were still of quite small magnitude, were not progressive with time, and were not indicative of an unstable condition.

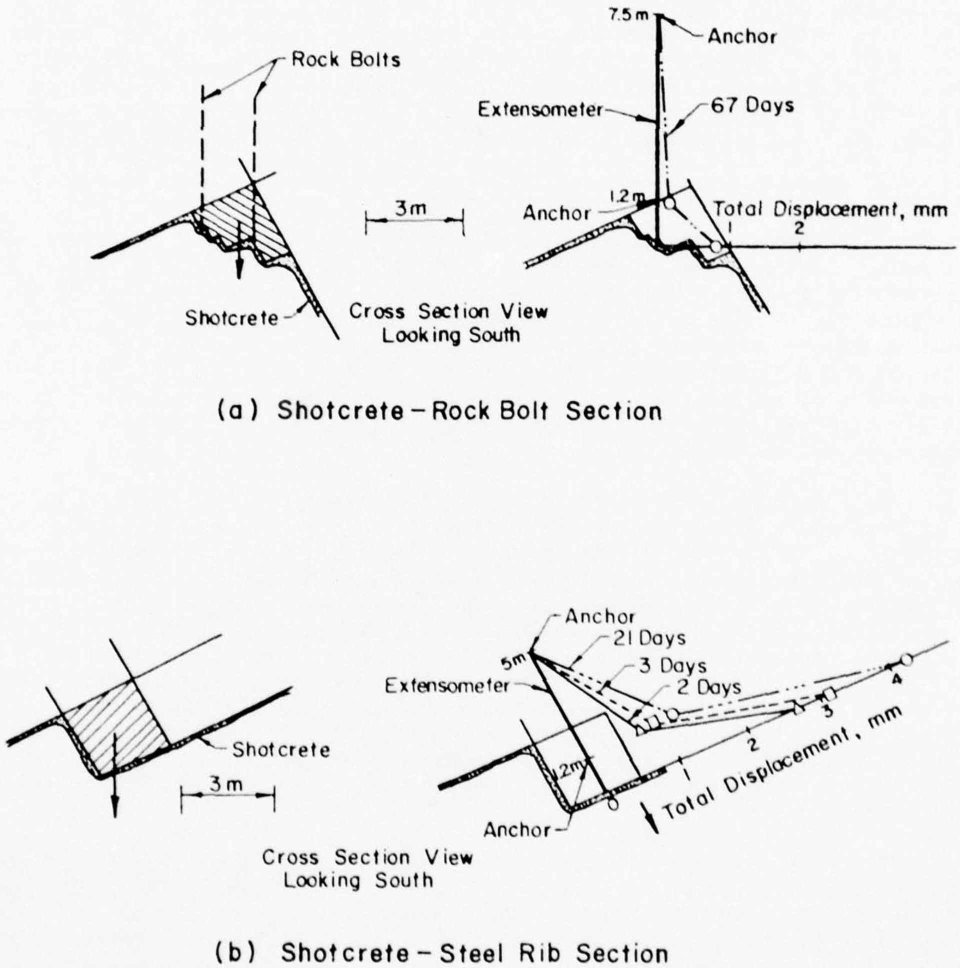

FIG. 5: DISPLACEMENTS IN ROCK TUNNEL, WASHINGTON, D. C. (MAHAR, GAU, AND CORDING, 1972).

FIG. 5: DEPLACEMENT DANS TUNNEL ROCHE, WASHINGTON, D.C. (MAHAR, GAU, ET CORDING, 1972).

In other portions of the tunnel, steel ribs were installed after a thin (50 mm) layer of shotcrete was applied. The rock displacement in the crown of one of the steel rib sections was 4.3 mm, most of which was a result of loosening of a 2-meter-wide rock block which settled onto the steel rib (Fig. 5b). Such movements usually resulted in some cracking of the shotcrete, but the steel ribs in these sections were still capable of supporting the loosened rock blocks, thus the measurements did not indicate an unstable condition.

The typical displacements established for the 10-meter wide tunnel served as a basis for evaluating much larger displacements (12 mm) which developed at a later date in the crown of a narrower but higher tunnel. The severity of the movement was also indicated by the large volume of rock involved in the displacements and the high rates of displacement which continued with no further excavation in the vicinity of the extensometers. These displacements were stopped by placing additional support in the tunnel. Modified excavation and support procedures were then used throughout the remainder of the tunnel. Extensometers in the remainder of the tunnel showed displacements which were of the same order (2.5 mm) as the stable displacements which had been previously observed in well-supported sections of the 10-meter-wide tunnel.

4.3 Rate of Displacement

Displacement rates should be examined closely when evaluating the stability of a tunnel. Sudden increases in the rate of rock movement which are larger than would be expected for the increment of excavation carried out in the vicinity of the extensometer may provide an early indication of an unstable condition. High rates of movement which are unrelated to excavation or which continue after the face has advanced well beyond the extensometer location may also indicate an unstable condition. One of the best means of evaluating such rates of movement are to compare them with rates previously observed in portions of the tunnel which were well-supported and where the displacements ultimately stopped.

In Figure 6 displacement of the wall of the chamber is plotted versus time for three extensometers in one of the 28-meter-wide chambers at the Nevada Test Site. Extensometer C, in the side of the chamber, exhibited a total displacement of 8 mm, which was approximately equal to the calculated elastic modulus. After excavation was completed, its rate of displacement was 0.001 mm per day. Extensometer A at the top of the chamber had a total displacement of 25 mm, 3 times more the calculated elastic displacement. Most of the movement resulted from opening of a crack behind a 1-meter-thick slab at the surface of the chamber. After excavation was completed, the rate of displacement for extensometer A was still high (approximately 0.06 mm/day). Even though the rate appeared to be decreasing gradually with time, it appeared that loosening and cracking of the shallow slabs would continue and might cause loss of bearing beneath the bearing plates of the tensioned bolts. Gunite was placed over the rock surface to prevent further cracking.

On the planar face of the chamber (extensometer D), high rates of movement developed during excavation and continued after excavation was halted. In August the high displacement rates were reduced, but not stopped, by placing some additional rock bolts in the area where the movements were occurring. In early September, further excavation took place and the displacements accelerated. The rate was not reduced until a large number of rock bolts were added on the plane face of the cavern. Within two days, the displacements had stopped.

4.4 Volume of Rock Involved in the Displacement

Both the depth at which the rock mass is loosening and its areal extent must be known in order to determine the severity of the problem and the remedial measures to be taken. Figure 4 illustrates well the use of the multiple position extensometer (or a cluster of single-position extensometers) in determining the depth of movement. Movements such as those illustrated by Figure 4c were recorded at depth by five multiple-position extensometers spaced over a 15- by 15-meter area of the chamber wall. (The displacement-time relation is illustrated by extensometer D in Fig. 6). The stability of a large volume of rock was in question, and rock bolts long enough to anchor behind

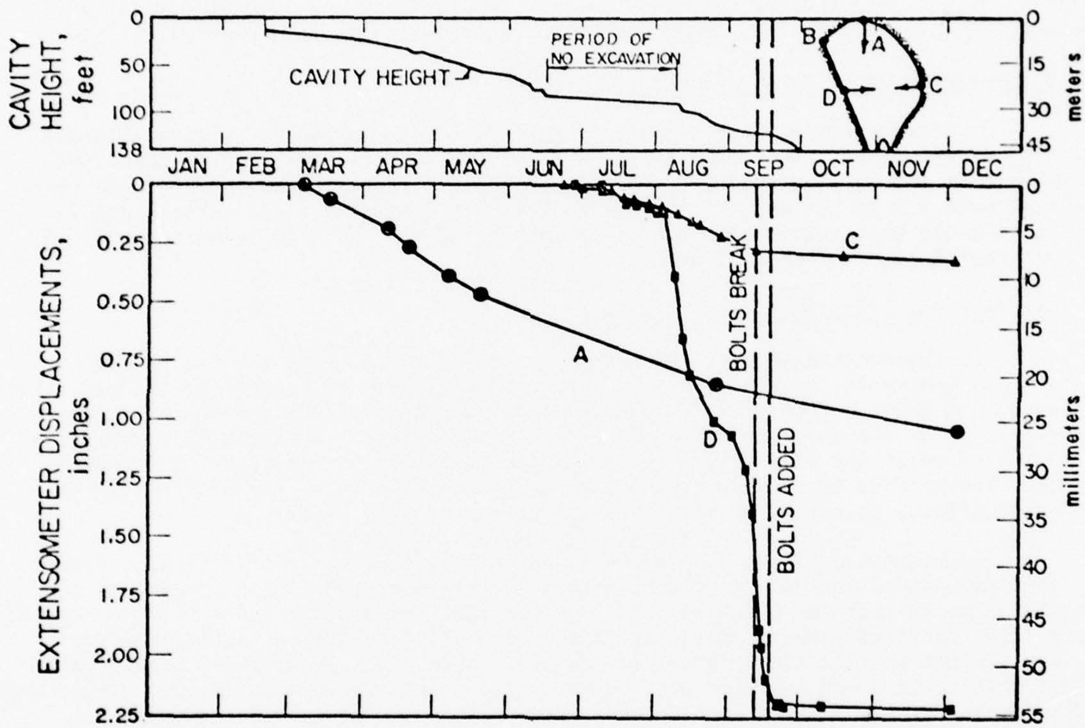

FIG. 6: DISPLACEMENTS AND EXCAVATION PROGRESS, CAVITY II, NEVADA TEST SITE (CORDING, HENDRON, AND DEERE, 1972).

FIG. 6: DÉPLAÇEMENTS ET PROGRÈS D'EXCAVATION CAVITÉ II, NEVADA TEST SITE (CORDING, HENDRON, ET DEERE, 1972).

the moving zone were required. In Figure 4b, the large movements were shallow and did not occur for all extensometers in the crown. Remedial measures were required only to stabilize a small volume of rock between the rock bolts.

Extensometers should be long enough to extend beyond the potential zone of movement. In the underground Machine Hall at Morrow Point Dam, 58 mm displacement took place on one wall of the chamber. The wedge was so large that a 15-meter-long extensometer on the wall of the cavern registered no displacement; it was located entirely within the moving wedge. The movements were detected by precise survey measurements in the chamber (Dodd, 1967).

If large displacements are allowed to continue with time, the depth at which the movements take place will increase. Figure 7 illustrates extensometer displacements in the crown of a chamber in horizontally bedded, sedimentary rocks. With time, as the displacements continued beyond the elastic displacement, separation took place along bedding planes further above the crown of the chamber. Progressively deeper movements have also been observed along joints in blocky ground. In such cases, if support is delayed, the ultimate support capacity required to stabilize the movements will be greater than the capacity that would have been required had support been placed sooner.

4.5 Displacement Capacity of the Support System

Observed displacements should not exceed the displacements which will cause distress or failure of the support system. Shotcrete has been observed to crack in the tunnels of the Washington, D. C. Metro when the differential movement between rock blocks exceeds 1.2 to 2.5 mm. At the Nevada Test Site, bearing plates dished and 11 rock bolts broke when rock displacements approached 50 mm. (The bolts were tensioned, but not grouted, over a 7-meter length.)

4.6 Displacement Capacity of the Rock Mass

Displacements should not exceed the capacity of the rock mass to maintain its strength and coherence, unless the support system is capable of supporting the resulting increased rock loads. Rock strength along joints decreases with displacement as irregularities on the joint surface are sheared or overriden. When the 50 mm displacements took place on the wall of Chamber II at the Nevada Test Site (Fig. 4c), fresh fractures accompanied by audible rock noise formed on one side of the wall where there were no continuous joints along which the failure could take place.

The magnitude of the displacements which will cause loss of strength of the rock mass depends on the amount of displacement it takes to override or shear off the irregularities so that the block will fall out or that residual strength will be reached on the joint surfaces. Where joints are planar and slickensided and sufficient joint sets are present to form blocky rock, the displacements required to cause failure will be less than those where surfaces are irregular and joints are discontinuous. This range of displacements is estimated to be approximately 2.0 to 50 mm in Washington, D. C.

4.7 Supplemental Observations

Supplemental observations of support load, support distress, and rock movement will aid the interpretation of displacement measurements. Some of these observations are outlined below:

a. Opening of joints or movement of rock blocks. Such observations can be made visually or with a tape measure or survey. Open cracks in boreholes can be inspected with a stratascope. Rock displacements are also indicated by offsets in open boreholes drilled prior to the time of the rock movements. Any loose slabs in the vicinity of the extensometer should be noted.

b. Mapping of joints, shear zones, and other geologic features which could cause movement. Observation of overbreak and rock loosening along the joints and shear zones will aid in evaluating the significance of these features.

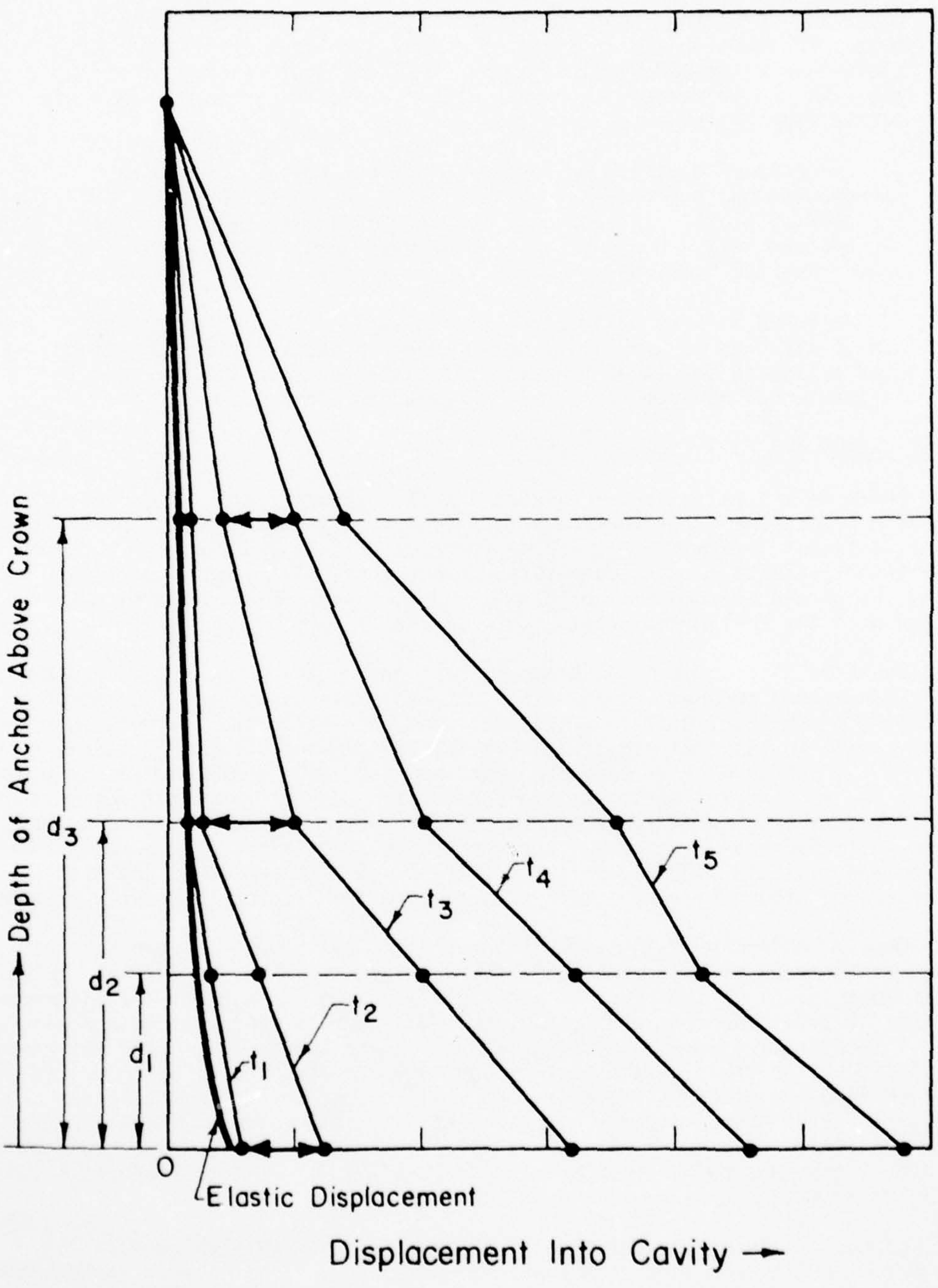

FIG. 7: PROGRESSIVE DISPLACEMENTS WITH DEPTH (CORDING, HENDRON, AND DEERE, 1971).

FIG. 7: DÉPLACEMENTS PROGRESSIFS AVEC PROFONDEUR (CORDING, HENDRON, ET DEERE, 1971).

c. Crack surveys in shotcrete. The width, length, and relative movement of the crack should be measured with time, and the thickness of the shotcrete in the vicinity of the crack determined.

d. Evidence of distress or displacement of steel ribs and timber blocking. Crushing, bending, or loosening of the timber should be noted. Distortion and twisting of rib sections and opening of butt plates can be measured with a tape. Deflection or settlement of the ribs can be measured by survey. Closure of ribs can be measured with a tape extensometer or a tape measure.

e. Evidence of distress or loosening of rock bolts. Tensioned, non-grouted bolts should be checked for loosening of the bolts at the bearing plate. Increases in load are indicated by dishing of the bearing plate, bending of the bolt head, or breaking of the bolt. Grouted bolts do not exhibit such distress.

f. Measured strains or loads on the support system. Strain gages can be attached to steel ribs or embedded in shotcrete or concrete. Load cells can be placed beneath posts. Gages which will be stable in the tunnel environment should be selected.

5. PRESENT TRENDS AND FUTURE DEVELOPMENTS

There is a trend toward increased use of instrumentation in tunnels. There will also be a trend toward an increase in the number of instrumentation programs which provide little useful information on the performance of tunnels. This second trend should not materialize if the instrumentation program is related to significant engineering design and construction problems, and is organized and carried out in close coordination with the design and construction staffs.

Borehole displacement measurements have proven to be the foundation of most successful measurement programs in tunnels. Extensometers as well as inclinometers have been increasingly used for monitoring stability and for evaluating the movements which could cause damage to adjacent structures. Extensometers are becoming a standard means of monitoring the performance of large chambers in rock, and should increasingly be used routinely by the engineering geologist and construction engineering staff as part of the overall observation program on large underground projects. There is a growing body of displacement data for both soft ground and rock tunnels which has been correlated with construction and geologic conditions in the tunnel. These data can now be used as a basis for planning other instrumentation programs and for interpreting their results.

Once an underground instrumentation program has been organized so that the significant performance will be measured, the major difficulty is to install, maintain, and protect instruments as construction takes place in the vicinity of the instrument installation. This is the time the most useful measurements are obtained and also a time when it is difficult to obtain access and to extend and protect leads for remote reading. If at all possible, extensometers and inclinometers should be installed prior to excavation from the ground surface or from small drifts which are not in the immediate vicinity of the excavation being monitored. Extensometers installed near the heading of the tunnel must be simple to install. Delicate assembly work should be accomplished in the shop rather than the field. Leads to electrical readouts should be easily spliced or extended without changing the zero.

Borehole extensometers and inclinometers are available which have the precision and stability for long-term measurement in rock. Inclinometer systems now appear to have the precision that will permit their use in measuring displacements around rock chambers or rock cuts. The inclinometer can be installed prior to excavation in boreholes behind the wall of a cut or chamber. As excavation proceeds, the lateral displacements of the wall are monitored.

Future developments in underground displacement instrumentation should be directed toward simpler installation methods, improved reliability of electronic equipment, improved protection of instruments from corrosion, moisture, and shock, and increased use of pre-packaged, plug-in units which can be replaced without losing the displacement zero. Since systems are already available which have sufficient repeatability and reliability for most tunnel applications, new instrumentation systems are not needed unless they have been thoroughly bench- and field-tested and promise to be more reliable, and more precise than those presently available.

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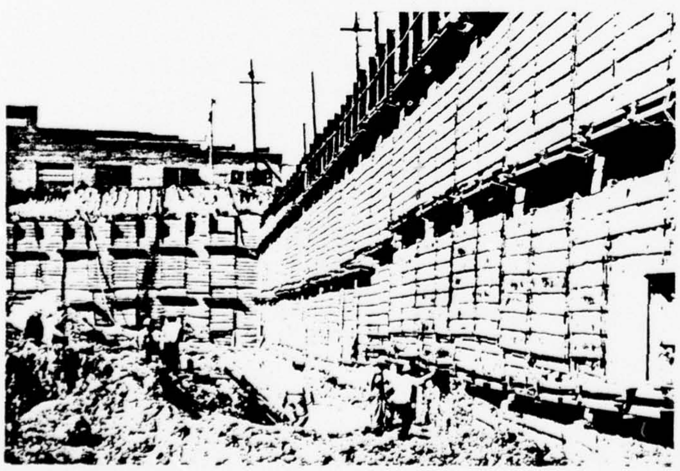
APPENDIX D

DYWIDAG ROCK AND SOIL ANCHORS

1¼" Dywidag threadbar soil anchors installed in a dry sand and gravel to tieback H-beam and concrete underpinning wall in 40 ft. deep excavation. Anchor design and installation by Richard Goettle Inc., Cincinnati, Ohio.



1⅜" Dywidag threadbar soil anchors installed in wet sand to tieback H-beam and wooden lagging system in 40ft. excavation. Anchor design and installation by Schnabel Foundation Co., Washington, D. C.

Dyckerhoff & Widmann, Inc.

DYWIDAG PRESTRESSED CONCRETE

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11526 SORRENTO VALLEY ROAD, SAN DIEGO, CA. 92121 • (714) 755-6787
Cable Address: Dywidag San Diego
2200 KENSINGTON CT., OAK BROOK, ILL. 60521 • (312) 986-5270

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DYWIDAG

The Dywidag Threadbar System is manufactured in the United States exclusively by Dyckerhoff & Widmann, Inc. Used world-wide since 1965, the threadbar system provides a simple, rugged method of efficiently applying prestress force to a wide variety of structural systems including posttensioned concrete, rock and soil anchor systems.

Available in ⅝", 1", 1¼" and 1⅜" nominal diameter, Dywidag threadbars are hot rolled and proof stressed alloy steel conforming to ASTM A 722-75.

The Dywidag threadbar has a continuous rolled-in pattern of threadlike deformations along its entire length. More durable than machine threads, the deformations allow anchorages and couplers to thread onto the threadbar at any point.

Exceeding the strength requirements of ACI 318-77, all Dywidag anchorages and couplers are designed to develop 100% of the guaranteed ultimate strength of the threadbar.

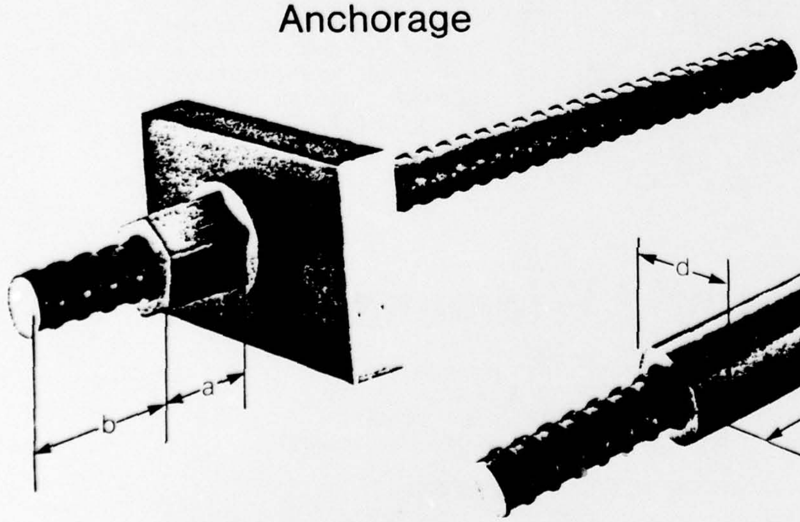
Conforming to the requirements of ASTM A 615-75, the threadbar deformations develop an effective bond with cement or resin grout. The continuous thread simplifies stressing. Lift off readings may be taken at any time, and the prestress force increased or decreased as required.

The anchor plate need not be perpendicular to the Dywidag threadbar. The curved surface of the anchor nut accommodates up to 5° misalignment. As much as a 25° misalignment of the threadbar with the bearing plate can be corrected by using a set of wedge washers with the anchor nut.

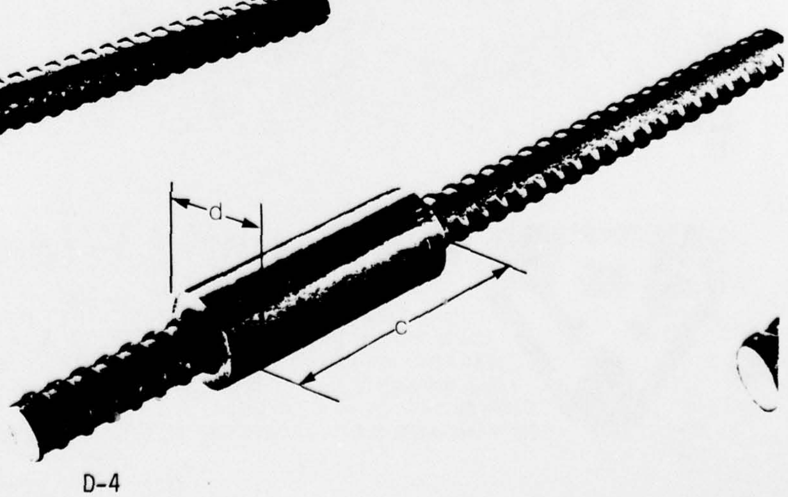
Available in mill lengths to 60', threadbars may be cut to specified lengths before shipment to the job site. Or where circumstances warrant, the threadbars may be shipped to the job site in mill lengths for field cutting with a portable friction or band saw. Threadbars may be coupled for ease of handling or to extend a previously stressed bar.

The Dywidag Threadbar System is used extensively in rock and soil anchor construction because of its versatility, strength, performance characteristics and off-the-shelf availability of most components.

Anchorage



Coupling



Threadbar Rock and Soil Anchors

Prestressing steel properties

Threadbar Diameter (inches)	Ultimate Stress (f_{pu}-ksi)	Cross Section Area (A_{ps}-inches²)	Ultimate Strength $f_{pu} A_{ps}$	Prestressing Force — (kips)			Weight** (lbs./ft.)	Minimum Elastic Bending Radius (ft.)
				$0.80 f_{pu} A_{ps}$	$0.70 f_{pu} A_{ps}$	$0.60 f_{pu} A_{ps}$		
⅝	157	0.28	43.5	34.8	30.5	26.1	0.98	26
1	150	0.85	127.5	102.0	89.3	76.5	3.01	52
1	160*	0.85	136.0	108.8	95.2	81.6	3.01	49
1¼	150	1.25	187.5	150.0	131.3	112.5	4.39	64
1¼	160*	1.25	200.0	160.0	140.0	120.0	4.39	60
1⅜	150	1.58	237.0	189.6	165.9	142.2	5.56	72

*Check on availability before specifying.
**Shipping weight may vary.

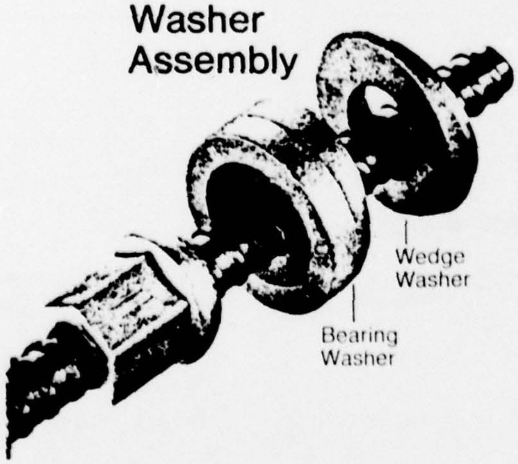
Steel stress levels

Dywidag threadbars may be stressed to the allowable limits of ACI 318-77. The maximum jacking stress (temporary) may not exceed 0.80 f_{pu}, and the transfer stress (lockoff) may not exceed 0.70_{pu}.

The final effective (working) prestress level depends on the specific application, installation procedure, stressing sequence, and the rigidity of the structural system. In the absence of a vigorous analysis of the structural system, $0.60 f_{pu}$ may be used as an approximation of the effective (working) prestress level.

Dywidag threadbars may be used individually or in multiples depending upon the magnitude of force requirements or upon drilling considerations.

Washer Assembly



Anchorage Details

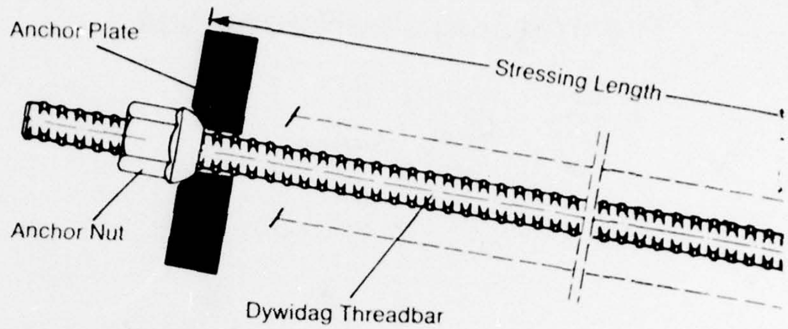
Threadbar Diameter (inches)	⅝	1	1¼	1⅜
Anchor Plate Size (inches)	3 x 3 x ¾ 2 x 5 x 1	5 x 5½ x 1¼ 4 x 6½ x 1¼	6 x 7 x 1½ 5 x 8 x 1½	7 x 7½ x 1¾ 5 x 9½ x 1¾
Nut Extension (inches) a	1	1⅞	2½	2¾
Min. Bar Protrusion (inches) b	2½	3	3½	4

Coupler Details

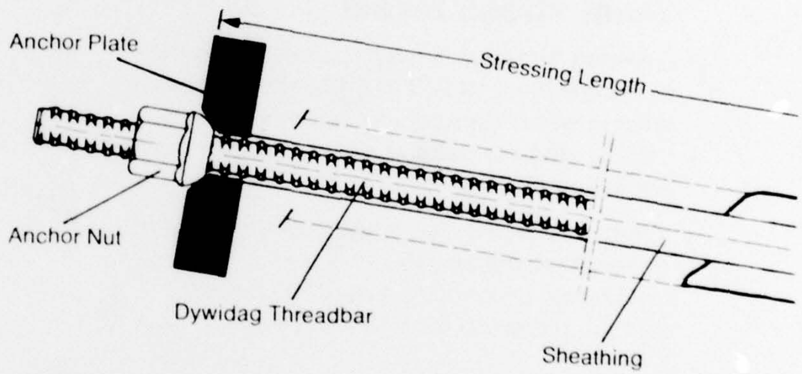
Threadbar Diameter (inches)	⅝	1	1¼	1⅜
Length (inches) c	3½	5½	6¾	8⅝
Diameter (inches) d	1⅛	2	2⅜	2⅝

DYWIDAG Anchor Details

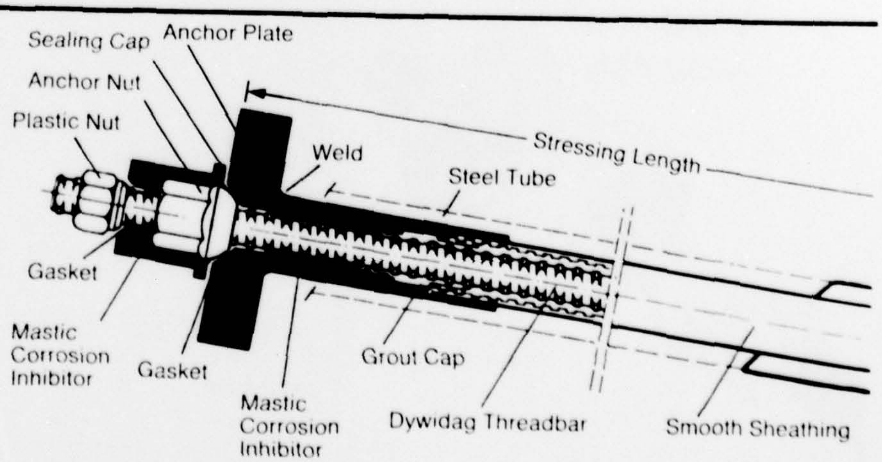
DYWIDAG Anchor without corrosion protection

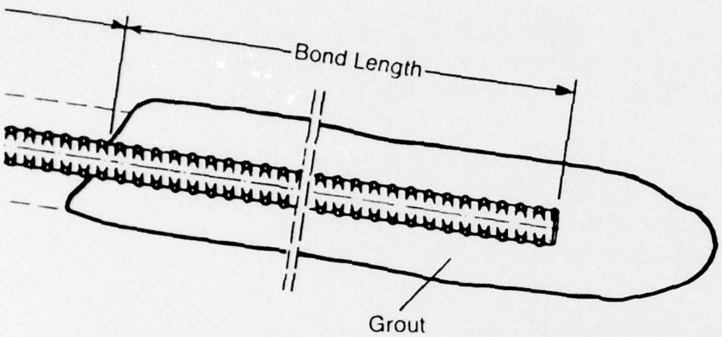


DYWIDAG Anchor with simple corrosion protection

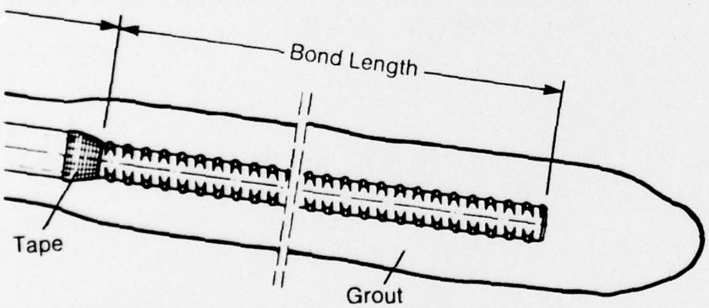


DYWIDAG Anchor with double corrosion protection

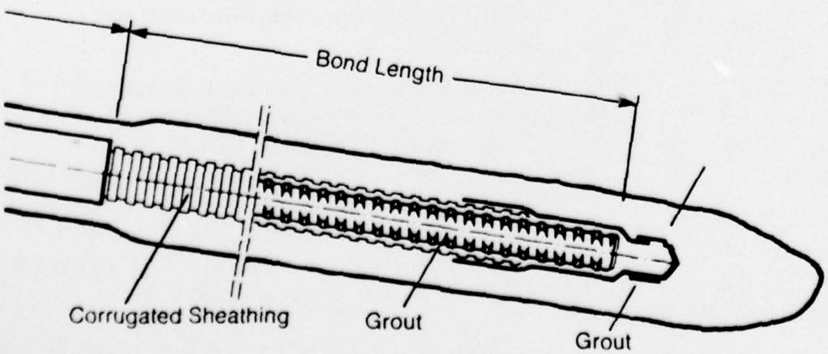




Maximum Anchor Diameter (in.)		
Threadbar Diameter	Without Coupler	With Coupler
5/8	3/4	1 1/4
1	1 1/4	2
1 1/4	1 1/2	2 3/8
1 3/8	1 5/8	2 5/8



Maximum Anchor Diameter (in.)		
Threadbar Diameter	Without Coupler	With Coupler
5/8	1	1 5/8
1	1 5/8	2 1/8
1 1/4	1 7/8	2 1/2
1 3/8	2 3/8	2 3/4



Maximum Anchor Diameter (in.)		
Threadbar Diameter	Without Coupler	With Coupler
5/8	1 1/8	1 5/8
1	2 3/8	2 1/2
1 1/4	2 3/8	3 1/8
1 3/8	2 7/8	3 1/8

Dywidag Anchor Design

The spacing, inclination, length and the load applied to each anchor depend on the local soil or rock conditions. The available drilling equipment and the structural capacity of the other support elements such as wales, lagging or a concrete retaining wall may dictate the capacity and configuration of anchors. A factor of safety of 1.5 to 2.5 should be utilized in anchor design.

For rock anchors, the shear stress on the rock socket perimeter is used to size the bond length. For soil anchors, the bond length is generally assumed on the basis of experience and site testing. Field testing should always be conducted to verify design assumptions.

Pull out tests verify that the bond capacity of the threadbar in grout exceeds the recommendations of ACI 318-77. The threadbar grout interface does not control the bond length. Bond in cohesive soils can be considerably increased using the Dywidag Postgrouting System.

The stressing length depends on the assumed failure plane and/or the size of the rock or soil mass necessary to resist the anchor force. A minimum stressing length of 20 ft. is recommended, so that small movements in the retaining system will not result in a major loss of prestress force.

Dywidag Anchor Corrosion Protection

Unprotected Anchors

Unprotected anchors are recommended for temporary use only. The stressing length is unprotected while the bond length is covered with cement grout. Unprotected anchors may be subject to corrosion, however, the relatively large diameter of the Dywidag threadbar offers more corrosion resistance than smaller diameter prestressing steel elements.

Simple Protected Anchors

Simple protected anchors may be used for temporary anchors or permanent anchors in unaggressive rock or soil. A polyethylene sheathing covers the stressing length. For permanent anchors the threadbar is coated with a corrosion inhibitor before the polyethylene is installed. The bond length is covered with cement grout.

Double Protected Anchors

Double protected anchors are recommended for anchors with a long service life and for an environment where aggressive materials or stray electrical currents are expected.

A corrugated PVC sheathing is installed over the bond length and the stressing length of the anchor. The annular space between threadbar and PVC is fully grouted before the anchor is installed. To accommodate the elongation during stressing, a short length of threadbar is left free of the corrugated sheathing at the stressing anchor.

A smooth PVC sheathing is installed over the corrugated sheathing in the stressing length. This accommodates elongation during stressing. The PVC sheathing makes a slip joint connection with a steel tube welded to the anchor plate. The steel tube is filled with a mastic corrosion inhibitor. A plastic cap filled with the corrosion inhibitor protects the anchor nut yet allows future stress adjustment.

The PVC sheathing is gas tight preventing intrusion of any corrosive substances. The grout around the threadbar provides a chemical corrosion protection by embedding the bar in a highly alkaline environment. The threadbar deformations minimize the size and control the distribution of any cracks that develop in the stressing length.

Dywidag Anchor Installation

Drilling

Selection of the drilling method depends on the number of anchors, the composition of the soil or rock, availability of equipment, and the required diameter of the hole. The selection of the tools and techniques should be left to the discretion of the drilling contractor where practical. The depth of the bore hole should be based on site tests.

The size of the bore hole should exceed the maximum diameter of the anchor by a minimum of 3/8 in. If centering devices are used, larger holes are required.

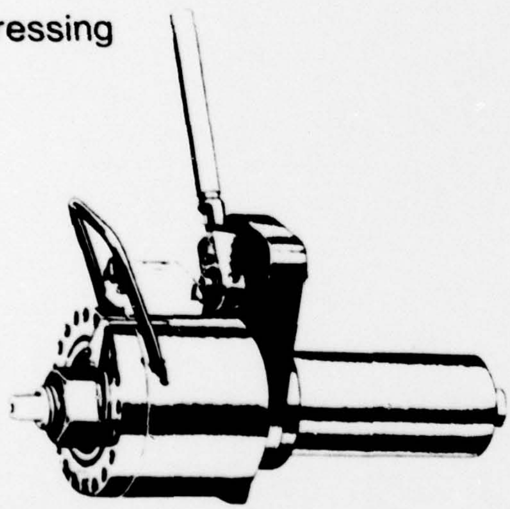
Grouting

For rock anchors, bore holes should be pressure tested to determine water leakage before the anchors are installed. Consolidation grouting, redrilling and retesting are required where water seepage is excessive.

After the anchor is installed in the bore hole, the bond length is grouted. Rock anchors and anchors in cohesive soils are generally grouted without pressure. Soil anchors in loose granular material are pressure grouted while the drill casing or auger is withdrawn.

Dywidag Postgrouting may be used for the installation of anchors in cohesive soils and non-cohesive soils. This technique permits additional grouting operations after the primary grout has cured. Using a series of valves in a preplaced grout pipe, grout can repeatedly be injected under high pressure. Regrouting displaces the previously injected grout and increases the anchor capacity.

Stressing



In stressing an electrically powered hydraulic jack with built-in socket wrench tightens the anchor nut. The jack fits over a pull rod designed to thread onto the threadbar extension protruding from the anchor nut. Elongation of the anchor can be measured directly or can be monitored by a counter on the jack. Hydraulic pressure is measured by a gauge on the hydraulic pump. Discrepancies of more than 10% between elongation and gauge reading should be investigated. Lift off readings should be taken to determine the applied prestress force. Movement of the structural system must be considered.

Testing

Prior to the installation of any production anchors, test anchors should be installed to verify all design assumptions including anchor length. Test anchors should be proof stressed to 80% of the guaranteed ultimate strength of the Dywidag threadbar. After 24 or more hours, readings should be taken on selected anchors to determine creep behavior.

All production anchors should also be proof stressed but the load need not be held for an extended period.

APPENDIX E

VSL PRESTRESSED ROCK ANCHORS

Introduction

Prestressed rock and soil anchors represent the application of the principle of prestressing to rock and soil. The increase in the use of prestressing in this application has been made possible by recent developments in the science of rock and soil mechanics. Both VSL Rock and Soil Anchors consist of prestressing tendons fitted at one end with a fixed anchorage, whose function is to transfer the anchoring force to the ground, and at the other end with a movable or stressing anchorage. These anchors may serve several purposes. Those listed below are typical:

(a) *Anchoring* of a structure to the ground (e.g., penstocks, above-ground pipelines, pylons, dams).

(b) *Prestressing* of a rock mass to improve its mechanical properties, thereby increasing its stability (e.g., prestressing an arched ceiling of an underground excavation to allow it to become self-supporting).

(c) *Anchoring and prestressing* rock or soil simultaneoususly, combining the functions of (a) and (b) (e.g., retaining wall).

The development of rock and soil anchorage techniques began over thirty years ago and found a wide application in Europe before being introduced in the U.S. The Swiss civil engineering firm, Losinger + Co SA, which has extensive experience in both post-tensioning and foundation engineering, has developed the VSL System for rock and soil anchors. The first application of VSL Rock Anchors dates back to 1957.

To date, the application of this system ranges from the simple repairing of a cracked structure to the prestressing of a self-supporting underground excavation.

Over 20,000 VSL Rock and Soil Anchors ranging from 25 to 1,100 kips in rock and from 25 to 145 kips in soil have been installed in more than 20 countries and on 6 continents.

VSL Prestressed Rock Anchors

The VSL Rock Anchor consists of a stressing anchorage, high-strength prestressing steel (270 ksi strand or 250 ksi wire), and a fixed anchorage bonded to the rock. The fixed anchorage is formed by intermittently spreading and constricting the strands or wires over the bonded length at the end of the tendon. When strand is used, spreading and constricting in the bond length may be omitted. The bond length is dependent upon the capacity of the tendon, the characteristics of the rock, and the drill hole diameter.

After placing the rock anchor, the fixed anchorage is grouted in place, bonding it to the surrounding rock (primary grouting). As soon as the primary grout has attained the specified strength, a VSL Stressing Anchorage Type E (or Type M for wire system) is fitted at the stressing end, and the rock anchor is stressed. The stressing can be accomplished in any number of stages, and the anchor force can be increased or decreased as the load-carrying requirements vary. Once the stressing is completed, a secondary grouting is accomplished to provide the steel with protection against corrosion.

For temporary or test anchors where the tendon must remain unbonded in order to permit subsequent checking of forces, the anchor is protected with a non-bonding anti-corrosive material. Although such materials may provide a suitable temporary protection against corrosion, their use is not recommended for permanent rock anchors.

VSL Rock Anchors are available in any length, ranging in working capacity from 25 to 1,290 kips and can be placed vertically, horizontally or sloped in an upward or downward direction.

Hongrin Power Station, Veytaux, Switzerland

Stabilization of underground excavation

Owner: Forces Motrices de l'Hongrin
Engineer: Compagnie d'Etude et Travaux Publics Lausanne
Contractor: Joint venture sponsored by Losinger + Co SA, Berne
Prestressing:
2200 VSL Rock Anchors of 25 to 370 kips
L = 12 to 70 ft.

Snettisham Project, Juneau, Alaska

Anchorage of outlet gate

Owner: U.S. Army Engineer District, Alaska Corps of Engineers
Engineer: U.S. Army Engineer District, Alaska Corps of Engineers
General Contractor: S. S. Mullen Construction Seattle, Washington
Prestressing:
14 VSL Rock Anchors of 130 to 470 kips
L = 50 to 100 ft.

Analysis of Safety Factor

An important feature of prestressed VSL Rock and Soil Anchors lies in the possibility of checking the load capacity of each individual anchor. Thus, the factor of safety against failure can be established. In the following paragraphs, the term "safety" is defined in detail.

Symbols

T_u = the ultimate tensile strength of the tendon steel
T_g = load in tendon causing the bond length of the steel to be pulled out of the grout (steel-grout bond failure)
T_b = load in tendon causing the fixed anchorage to be pulled out of the rock or soil (grout-rock or grout-soil bond failure)
T_p = proof load applied for a short duration to check the capacity of the anchor (normally 0.8 T_u)
T_w = working or design load to be taken permanently by the anchor (normally 0.6 T_u)
S_u = safety factor of tendon steel against failure under working load T_w
S_g = safety factor against the anchor pulling out of the grout under working load T_w
S_b = safety factor against the grouted anchorage pulling out of the rock or soil under working load T_w

Using the above notation, the safety factor of VSL Rock and Soil Anchors against tendon rupture at working force is:

$$S_u = \frac{T_u}{T_w} = \frac{1}{0.6} = 1.67.$$

Careful checks carried out on all tendon and anchorage steels assures their safety for each VSL Rock or Soil Anchor. VSL Anchorages will develop 100% of the guaranteed strength of the tendon steel.

When using a 10-foot bond length, the safety factor against the anchor pulling out of grout cylinder $S_g = \frac{T_g}{T_w}$ is known from testing to be significantly greater than the safety factor of tendon steel against failure.

On the jobsite, a check of the safety factor S_b against the anchorage pulling out of the soil may be performed. This is accomplished with a temporary test loading. This allowable test load T_p, however, is limited by the elastic limit of the prestressing steel. If the test load is attained,

then $S_b = \frac{T_p}{T_w} = \frac{0.8\ T_u}{0.6\ T_u}$ = at least 1.33.

This method of testing includes a consideration of the local ground conditions in the bond anchorage zone which are of the utmost importance and which often vary considerably. The value of the "safety factor" thus obtained against the anchorage pulling out of the soil or rock is, of course, a low limit. A method of determining the actual value of this safety factor involves the use of an anchorage with a reduced bond length whose pullout load T_b can be achieved by the test load, T_p. The pullout load of the test anchor can thus be measured, a safety factor established, and the bond lengths of the remaining anchors determined. This method establishes the actual value of the safety factor S_b for the rock or soil in the immediate vicinity of the test tendon. Judgement must be used in establishing the distance from this test anchor for which the safety factor is appropriate. The safety factor selected for design depends on such items as the following:

- knowledge of the actual rock or soil condition
- extent of the proposed monitoring and proof loading program
- life of the structure
- consequence of a failure
- extent of deformation which is tolerable without impairing the utility or serviceability of the structure

Determination of Bond Length

A. Rock Anchors

The bond length required for rock anchors can be determined using the pullout test described above or from rock samples. As a simplification, it can be assumed that the transfer of the force to the rock occurs along the surface of the cylinder-shaped bond length by means of uniformly distributed shear stresses τ_b. The testing procedure consists of placing the core sample vertically in the middle of a steel form and filling the surrounding space with grout. Subsequent to curing, the sample is pressed out and the bond strength between it and the grout measured.

The values for τ_b given below are excerpts from reports of laboratory tests carried out by Gribaldo, Jones & Associates in California.

	Ultimate Bond Strength τ_b in psi
1. Soft sandstone, poorly lithified, extremely friable, fine grained	53
2. Serpentine, moderately weathered, slickensided	225
3. Hard sandstone, lithified and well-cemented with calcite, fine grained, slightly friable	325
4. Tertiary limestone	400
5. Chalky sandstone	411
6. Basalt, moderately weathered with a few stringers of calcite	560

A safety factor must be applied to bond strength values obtained in the manner described above, taking into consideration the points mentioned in the previous section, and a bond length can then be established. The above values are, of course, typical of the rock only in the location from which it was obtained, and each new site should be investigated individually.

B. Soil Anchors

The conditions are quite different with soil anchors. During the primary grouting, a grout bulb is formed in the anchorage bond zone, the size of which depends on the compressibility and permeability of the ground and the grouting pressure. Tests have demonstrated that the pullout load for VSL Soil Anchors in non-cohesive soils can be represented for a given set of typical conditions by the following formula which can be used as a first estimate in design:

$$T_b = N \times L \tan \Phi$$

where N = 27 to 40 kips per foot of fixed anchorage
L = bond length
Φ = angle of internal friction

Selection of Anchor

The following points should be considered in projects using rock and soil anchors:

— determination of the anchorage direction and force to establish equilibrium
— selection and design of the anchors
— establishing the stressing sequence

When selecting the anchor, consideration should be given to the structure, the forces to be transferred and the local load-carrying capacity of the ground. Retaining walls, for example, may be stabilized by either many small or a fewer number of large anchorage forces. The dimensions of the wall itself may be permitted to regulate or may be regulated by the size and spacing of the anchors. The most economic design will require consideration of all factors. The technical staff of the VSL Corporation is at the service of designers desiring assistance in such determinations.

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SELECTING ROCK AND SOIL ANCHORS

The following points are suggested for consideration when selecting a prestressing unit.

Working force

Loss of prestress

Allowable stresses in prestressing steel

Drill hole diameter

Bond length

1/2" dia. 270 ksi strands

The 1/2", 7-wire strand for prestressing application has an ultimate strength (f_s') of 270,000 psi and is produced and tested in accordance with the requirements of ASTM A 416 (latest revision). Physical properties of 1/2" strand are as follows:

Guaranteed Ultimate Strength	41,300 lb.
Yield strength (at 1% extension)	35,100 lb.
Approx. modulus of elasticity	27,000,000 psi*
Min. elongation at rupture	3.5% in 24 inches

* (This figure may vary slightly for different manufacturers)

Unit	No. of strands	Steel area	Weight	Diameter of drill holes in inches (approximate)			Max. temp. force 0.8 f_s'	Initial force 0.7 f_s'	Working force 0.6 f_s'
				Rock Anchors (1) in.	Soil Anchors (2) in.				
		sq. in.	lb./ft.		Bulb	Augered	kips	kips	kips
ERS-1	1	0.153	0.525	1-1/2	1-3/8	6	33.0	28.9	24.8
ERS-3	3	0.459	1.575	2-1/2	1-3/4	6	99.1	86.7	74.3
ERS-6	4	0.612	2.100	2-1/2	1-3/4	12	132.2	115.6	99.1
	5	0.765	2.625	3	3	12	165.2	144.5	123.9
	6	0.918	3.150	3	3	12	198.2	173.5	148.7
ERS-9	7	1.071	3.675	3	3	18	231.3	202.4	173.5
	8	1.224	4.200	3-1/2			264.3	231.3	198.2
	9	1.377	4.725	3-1/2			297.4	260.2	223.0
ERS-16	10	1.530	5.250	4			330.4	289.1	247.8
	11	1.683	5.775	4			363.4	318.0	272.6
	12	1.836	6.300	4-1/2			396.5	346.9	297.4
	13	1.989	6.825	4-1/2			429.5	375.8	322.1
	14	2.142	7.350	5			462.6	404.7	346.9
	15	2.295	7.875	5			495.6	433.6	371.7
	16	2.448	8.400	5			528.6	462.6	396.5
ERS-19	17	2.601	8.925	5			561.7	491.5	421.3
	18	2.754	9.450	5-1/2			594.7	520.4	446.0
	19	2.907	9.975	5-1/2			627.8		
ERS-28	20	3.060	10.500	5-1/2			660.8	578.2	495.6
	21	3.213	11.025	5-1/2			693.8	607.1	520.4
	22	3.366	11.550	8			726.9	636.0	545.2
	23	3.519	12.075	6			759.9	664.9	569.9
	24	3.672	12.600	6			793.0	693.8	594.7
	25	3.825	13.125	6			826.0	722.7	619.5
	26	3.978	13.650	6			859.0	751.7	644.3
	27	4.131	14.175	6			892.1	780.6	665.6
	28	4.284	14.700	6			925.1	809.5	693.8
	29	4.437	15.225	6-1/2			958.2	838.4	718.6
	30	4.590	15.750	6-1/2			991.2	867.3	743.4
	31	4.743	16.275	6-1/2			1024.2	896.2	768.2
ERS-52	52(3)	7.956	27.30	9			1718.5	1503.7	1288.9

(1) The diameter of the drill holes is given for information only. They may vary according to the type of anchor, characteristics of rock (loading capacity) and drilling equipment available.

(2) When soil conditions demand that the drill hole be lined, then these dimensions correspond to the internal diameter of the casing.

(3) Intermediate and larger units available upon demand.

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